PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
SEPTEMBER 1952

ORDINARY MEETING

18 March, 1952

ALLAN STEPHEN QUARTERMAINE, C.B.E., M.C., B.Sc.(Eng.),
President, in the Chair

The President invited the Members present to stand while he read the following resolution on the death of Sir Roger Hetherington, which had been passed by the Council that day:

> "That the Council record the very deep regret with which they have learned of the death of Sir Roger Gaskell Hetherington, and desire to convey to Lady Hetherington and the members of his family an expression of sincere sympathy in their bereavement."

Sir Roger Hetherington, the President observed, had been a Member of the Institution for more than 50 years. He had been elected a Member of Council in 1937, a Vice-President in 1946, and had reached the Presidential Chair in 1947. For nearly 14 years Sir Roger had taken a close and active interest in the Council and Committee work of the Institution. He had also taken a prominent part in many movements which added to the prestige and authority of the Institution, and the Institution owed much to his wise counsel and unwearied service. His colleagues desired to record the value that they had attached to his service to the Council and the sense of loss which they, in common with the whole of the Institution, had sustained by his death.

The Council reported that they had recently transferred to the class of

Members

NATHANIEL HARVEY HUNT.
THOMAS LESLIE LOWE, B.Sc. (*Wales*).
OSWALD STEVENS NOCK, B.Sc. (Eng.) (*Lond.*).

and had admitted as

Students

JOHN NORMAN ALLISON.
PANCHANAYAGAM ARULIAH.
RALPH WILLIAM ASTORGA, B.Sc. (*Manch.*)
ANTHONY PATRICK ATHAWES.
NORMAN RICHARD AYLIFFE.
KENNETH MARK BALDWICK.
PETER BODEN.
GEOFFREY BOOTH.
AUBREY HERBERT BRAIN.
THOMAS DENNIS BULMER.
DAVID BRYAN BURROW.
GEOFFREY COLLOFF.
JOHN HAWTHORNE CORTEEN.
JOHN CRAN CRAWFORD, B.Sc. (*Glas.*).
DENIS PASCHAL CRONIN, B.E. (*National*).
JAMES WALTER FREDERICK CUNLIFFE.
ALAHAPPERUMAARACHCHIGE DHARMARANSI DIAS.
JOCELYN FRANCOIS SMUTS DU TOIT, B.Sc. (*Cape Town*).
PETER FREDERICK DYER.
NORMAN EDMONDS.
GEORGE VERNON FAWKES.
WILLIAM JOHN FLEMING.
PHILIP JACKSON FLETCHER.
DESMOND FOSTER SEPTIMUS GAMLEN.
JOHN IRVINE GLANVILLE.
GRAHAM DUDLEY GODDARD.
PETER GEORGE GOLDSBRO'.
STANLEY FREDERICK GOWER.
THOMAS DAVID GRUNDY.
ALLAN COOK BRUCE GUILD.
TERRY HALL.
DEREK ANTHONY HAMMOND.
GEORGE JOHNSTONE KERR HARRIS, B.Sc. (*Glas.*).
JOHN MICHAEL HARRISON.
RAYMOND HARRISON.
RICHARD CEDRIC HART-JONES.
FREDERICK JOHN HAWKINS.
NORMAN JOHN HOBBS.
JOHN PRICE HOPKINS.
DAVID JERMAN HUGHES.
JOHN HUGHES, B.Sc. (*St. Andrews*).
DENIS WILLIAM HUMPHREYS.
JOHN DILLON HUMPHREYS, B.A. (*Cantab.*)
HENRY BARRY HUMPIDGE.
JOHN HAMPDEN HYATT.
WILLIAM ANTHONY LUMLEY JAMES, B.A., B.A.I. (*Dublin*).
HAROLD GRAHAM JOHNSON.
RICHARD DESMOND JUKES.
THOMAS BLACKWOOD KIRK.
GORDON EVANS KNIGHT.
KEKI JEHANGIR LAWYER.
DUNCAN JONES LEACH, B.Sc. (*Manch.*).
JAMES ARMOUR LINDSAY.
ROBERT DAVID MCCLURE.
PATRICK JAMES MACKEY.
ROBIN MACK-SMITH, B.A. (*Cantab.*).
DAVID ERNEST MAYO.
ERIC WILLIAM MESSER.
JOHN DEREK METCALF.
GORDON STEWART MURRAY.
THOMAS WILLIAM ROBERTSON NEILL.
PATRICK TIMOTHY O'CONNELL, B.E. (*National*).
HARRY PADGETT.
PETER JOHN PARSONS.
DONALD PARTRIDGE.
ERIC IRVIN PENNINGTON, B.Sc. (*Eng.*) (*Lond.*).
THOMAS PONSONBY.
BASIL GRIFFITHS PREESE.
FRANK SLINGSBY PROCTOR.
RAYMOND DAVID RAWLINGS.
WILLIAM JOSEPH FERGUS RAY.
JAMES ALFRED READ.
ROBERT NICOL REVIE.
BERNARD PERCY REYNOLDS.
JOHN REGINALD RICHARDS.
DAVID EDGAR ROE, B.Sc. (Eng.) (*Lond.*).
GORDON VICTOR ROLLINSON.
DENNIS ROY SEAWARD.
COLIN JOHN SIEBERT, B.Sc. (Eng.) (*Lond.*).
GEOFFREY HARDWICK SIMONS.
PETER JOHN SKELDON.
GORDON RAYMOND SMITH, B.Eng. (*Liverpool*).
DAVID SPEDDING.
RONALD SWAN, B.Sc. (*Aberdeen*).
OTTO GEORGE TELLER.
JAMES REUBEN KEITH TRYNER.
BRIAN LESLIE UNDERWOOD.
JOHN LESLIE WELLINGTON.
CLIFFORD MICHAEL WILLIAMS.
DAVID HENRY WILLIAMS.
ROBERT SIDNEY WILLIAMS.
DEREK ROBERT WOLSTENHOLME.
JOHN WOODALL.

The Scrutineers (Mr F. P. Dath and Mr J. E. Duncan, Associate Members) reported that the following had been duly elected as

Associate Members

SAMUEL LOUIS ABBOTT, B.Sc. (Eng.) (*Lond.*).
JOSEPH GEORGE NORMAN BAILIE, B.Sc. (*Belfast*).
ARTHUR INVERDALE BARRY, B.Sc. (*Edin.*)
THOMAS WILLIAM NORMAN BEASANT, B.Sc. (*Glas.*).
PETER GRAHAM DESMOND BELL, B.Sc. (*Leeds*) (*Stud. I.C.E.*).
Major JOHN BROAD, R.E.
BRIAN ALEXANDER COULTER, B.Sc. (*Belfast*).
DOUGLAS DANIEL GATE, B.Sc.Tech. (*Manchester*).
JOHN GAULT, B.Sc. (*Belfast*).
JOSEPH OLIVER GIBSON, M.A. (*Cantab.*). (*Stud. I.C.E.*).
NORMAN ERIC GLOVER, B.Eng. (*Liverpool*).
ERIC CROSSFIELD GORDON.
ROBERT GRAY.
THOMAS HINDLEY.
DENIS MALCOLM HULBERT.
IBRAHIM MOHAMED IBRAHIM.
KENNETH JAMES IVES, B.Sc. (Eng.) (*Lond.*) (*Stud. I.C.E.*).
KENNETH ROY JONES, B.Sc. (*Glas.*), (*Stud. I.C.E.*).
PARAKRAMA DUNCAN WESTBROOK KULARATNE, B.Sc. (Eng.) (*Lond.*) (*Stud. I.C.E.*).
CARL ARTHUR LONG.
HERBERT STANLEY MAYO, B.Sc.Tech. (*Manchester*).
ALBERT EDWARD NAYLOR, M.Eng. (*Liverpool*) (*Stud. I.C.E.*).
FRANK EDWARD OLDROYD, B.Sc. (*Manchester*) (*Stud. I.C.E.*).
ROBERT JOHN ORMAN.
DUDLEY JOHN HENRY PAYTON, B.Sc. (Eng.) (*Lond.*).
VICTOR HAMMETT PONTIN, B.Sc. (Eng.) (*Lond.*) (*Stud. I.C.E.*).
JOHN REEVES STANLEY (*Stud. I.C.E.*).
GEOFFREY CLIFFORD TAVERNER, D.F.C., B.Sc. (*Eng.*) (*Lond.*).
JAMES ORCHISON THORBURN, B.Sc. (*Glas.*) (*Stud. I.C.E.*).
GEORGE GREENWOOD WALKER.
RONALD WILLIAMS, B.A. (*Cantab.*).
WILFRED HENRY WILSON.
WILLIAM JOHN LAUNDON WISDISH, (*Stud. I.C.E.*).
THOMAS MCFEAT YOUNG (*Stud. I.C.E.*).

The following Paper was presented for discussion and, on the motion of the President, the thanks of the Institution were accorded to the Author.

Paper No. 5771

"The Most Recent Dams by the 'Società Adriatica di Elettricità (S.A.D.E.)' in the Eastern Alps"

by

Dott. Ing. Carlo Semenza

SYNOPSIS

The Paper describes the characteristics and methods of construction of the latest dams for hydro-electric plants in the Venetian Region by the Società Adriatica di Elettricità (S.A.D.E.). The description consists of:—

(1) The dam on the River Lumiei at Maina di Sauris.
(2) A general outline of the Piave-Boite-Vajont scheme.
(3) The dam on the River Piave at Pieve di Cadore.
(4) The Val Gallina dam.
(5) The dam on the River Boite at Valle di Cadore.
(6) A few words on the projected dams in the gorge of the River Vajont and on the River Maè.

For each dam are described:

(i) the ground conditions of the locations;
(ii) the data concerning dams and water reservoirs;
(iii) an outline of the design and studies carried out;
(iv) the preparation of the appropriate type of concrete;
(v) a list of the measuring and controlling apparatuses.

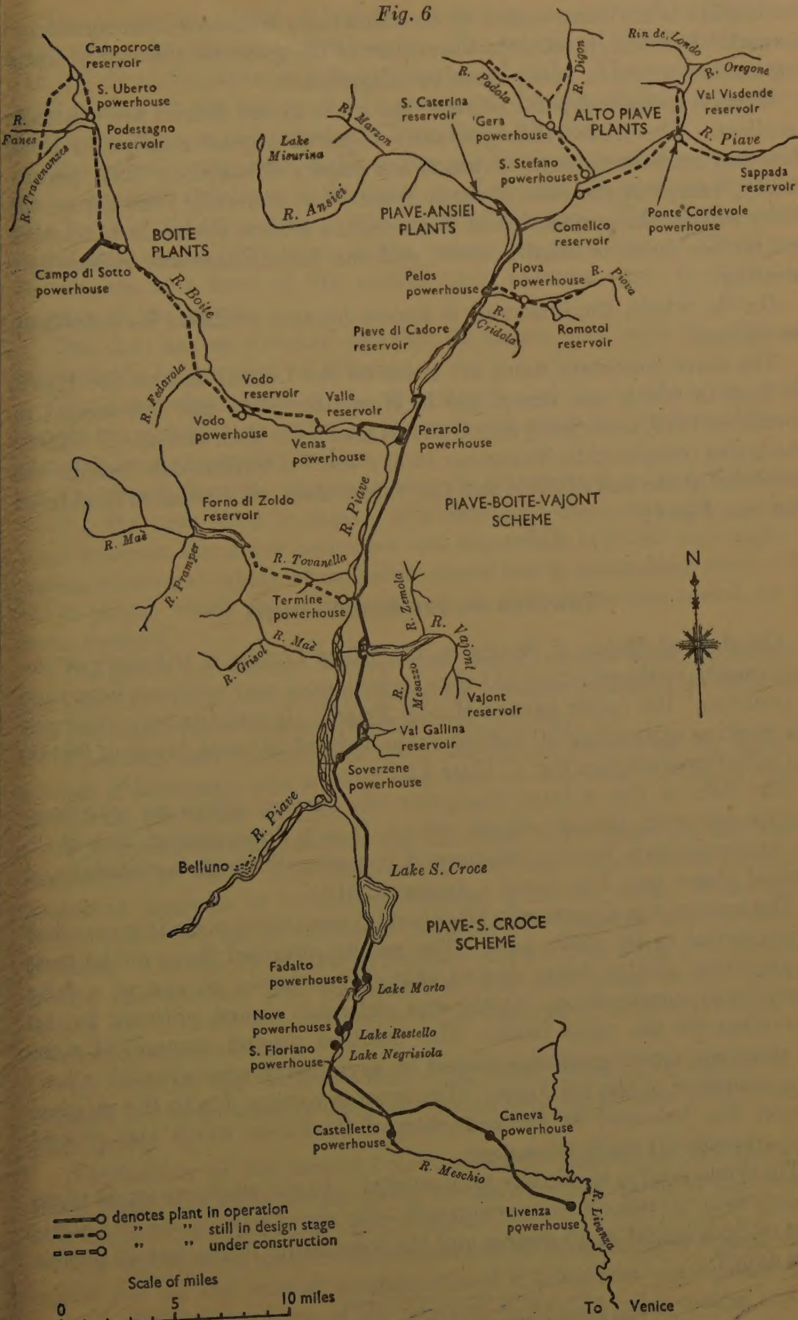
Introduction

In countries with hydrological conditions such as those which obtain in Italy, the problem of storage, and consequently of dams, is of primary importance in the design of hydro-electric installations.

The Author has been, for about 25 years, chiefly responsible for the new hydro-electric schemes and especially for the general planning and building of the civil engineering works of the S.A.D.E., which now distributes yearly about 2½ thousand million units in fifteen provinces, including the whole of Veneto, Venezia Giulia, Bologna, and Romagna as far as Rimini.

The region of the Venetian Alps and Prealps, where the plants are located, is characterized by the prevalence of limestone in its various geological ages. The sites for dams often consist of deep narrow gorges where simple arch or dome-shaped dams are the most suitable. However, since these gorges generally stretch for a considerable distance upstream,

Fig. 6



THE NORTH-EASTERN PIAVE HYDRO-ELECTRIC SCHEME

the heights of the dams exceed those which have until now been considered normal. In effect, if only the lower part of the gorge be closed, the water impounded would in many cases be insignificant.

From a geological point of view, the rocks in the district are generally excellent. The Author has gradually acquired a great respect for these Venetian limestones, most of which are mainly dolomitic. Their permeability, except in the case of carsic limestones, generally runs across the valley so that the permeation of water from a section upstream to another one further down is usually small, and can be substantially reduced by grouting.

On the whole, the limestone is honest because it shows its defects on the surface.

The more important dams are those on the Lumiei and on the Piave, the first of which was completed in November 1947 and the second in November 1949, both being tested at full load; the dam at Valle di Cadore which was completed in November 1950; the Val Gallina dam, which was completed at the end of 1951, and the Vajont dam, which was to be started soon (see *Fig. 6*).

The Dam on the River Lumiei

The dam at Maina di Sauris on the River Lumiei, a tributary of the Tagliamento in the province of Udine, is built on a very compact limestone ("Ladinico," the upper part of Middle Trias). It constitutes a reservoir of a working capacity of 70 million cubic metres* between levels of 980·00 and 905·00 metres (see Fig. 1, Plate 1).

The dam is like a huge triangular tile thrown across an enormous rocky gorge with almost vertical sides. It is a dome-shaped thin shell, the upstream face having pronounced curvature in both horizontal and vertical planes (see *Figs 4 and 5*, facing p. 512).

The section was chosen only after careful researches, since it was rather asymmetrical in its lower part. The results obtained from model-tests on preceding similar dams showed that it was better to aim at a fully symmetrical construction, which was obtained almost entirely by an excavation on the right bank. This solution proved economical since the abnormal twisting stresses caused by asymmetry were avoided, allowing a reduction of the thickness. The greater volume due to the increase of span was balanced by the reduction of thickness, which also proved to be an essential static advantage.

The dome itself is absolutely symmetrical. In the whole structure there are only two deviations from symmetry—the upper part of the left abutment up to a height of 15 metres and the foundation in the last 12 metres of its depth.

* Conversions are given in the Appendix, p. 535.

The maximum height of the dam from high-water level to the lowest point of the foundation is 134 metres. The top chord is 130 metres long and the chord/height ratio is therefore about 1 to 1. The crown-thickness is 16 metres at the base and 3·15 metres at the top, whilst at the abutments the increase of thickness is from 20 to 30 per cent.

The average curvature-radii of the geometric axes of the elementary arches conform strictly to high-degree parabolic laws from about 20 metres at a level of 854·00 metres to 78 metres at the top. The angular width varies from about 100 degrees at the base to the maximum of 128 degrees 30 minutes at a level of 910 metres, then narrows again to about 115 degrees at the crown.

The dam has an overflow spillway, 50 metres long, with five openings between the bridge piers on the crest.

The particular shape of the crown-arch and its overhanging profile causes the overflowing water to fall into the river bed at a considerable distance from the toe of the dam.

The thrust from the dome perimeter is transmitted to, and distributed within, the rock faces by means of a reinforced-concrete saddle, and the joint between the dome perimeter and the saddle is made watertight by means of several layers of asphaltic sheets and a staunching-piece. The other joints, which are at intervals of 15 metres are also sealed by the same method.

The joints are mainly vertical and radial, except when approaching the perimeter-joint, where their direction changes gradually so as to meet the perimeter-joint approximately at right angles.

Upstream and downstream faces are lightly reinforced by a double system of horizontal and vertical steel bars, at intervals varying in accordance with the stress values.

The mathematical analyses for the dam were based on the Italian regulations of 1931. The dam was divided into independent horizontal arches at intervals of 10 metres, starting from the crest. These arches were considered to be subjected to hydraulic-pressure and to thermal stresses. The theory of the hyperstatic elastic arch fixed into the abutments was applied, and consideration was also given to the shearing effects which could not be overlooked in the case of arches having a high thickness/length ratio.

By varying the thickness and using small radii of curvature it was possible to avoid excessive thickness and to keep the stresses within reasonable limits.

Verification of the arches below + 900·00 metres by the theory of thick rings, was quite simple owing to the very low value of the ratio of average thickness to radius of curvature.

The compressive stresses do not exceed 50 kilograms per square centimetre, nor do the tension stresses exceed 8 kilograms per square centimetre. The total volume of the concrete is 100,318 cubic metres and the

volume of the excavations is 63,000 cubic metres—a very high figure in comparison with that of the concrete.

The cross-section of the dam looks unusually thin (see Figs 2, Plate 1) but the structure gives a real sense of confidence. With due consideration to the features of the dam, model tests and design computations were directed by Professor Guido Oberti at the Polytechnic School of Milan.

It is the Author's opinion that the assistance of model-experiments in the development of dam design is becoming more and more necessary, in order to obtain timely confirmation of the direction and distribution of the stresses and a clear comprehension of the actual static behaviour of the structure. The model is indeed a calculating machine; it supports intuition, and most engineers will recall how frequently analysis deceives intuition and not *vice versa*.

On the rocky right-hand wall, slightly downstream, a deep couloir intersecting the hypothetical continuation of the horizontal arch of the dam was filled with concrete. Later on, its contact surfaces were deeply grouted. This provision, which required an extra quantity of 3,903 cubic metres of concrete of a lower quality than the one used for the dam, was found to be much less expensive than the displacement upstream of the whole of the right abutment which would have been necessary to avoid the hypothetical intersection.

The dam required concrete of very high quality and, in accordance with the Italian dam regulations, it had to attain a compressive strength of more than 350 kilograms per square centimetre after 90 days. The aggregate used had to be carefully graded, and mixed with a special cement, in order to reach the necessary imperviousness and to resist the action of the slightly sulphurous water of the Lumiei; for that reason, pozzolana was added to the clinker in a proportion of 1 : 3.

After long studies, a ferro-pozzolanic cement of high resistance was chosen.

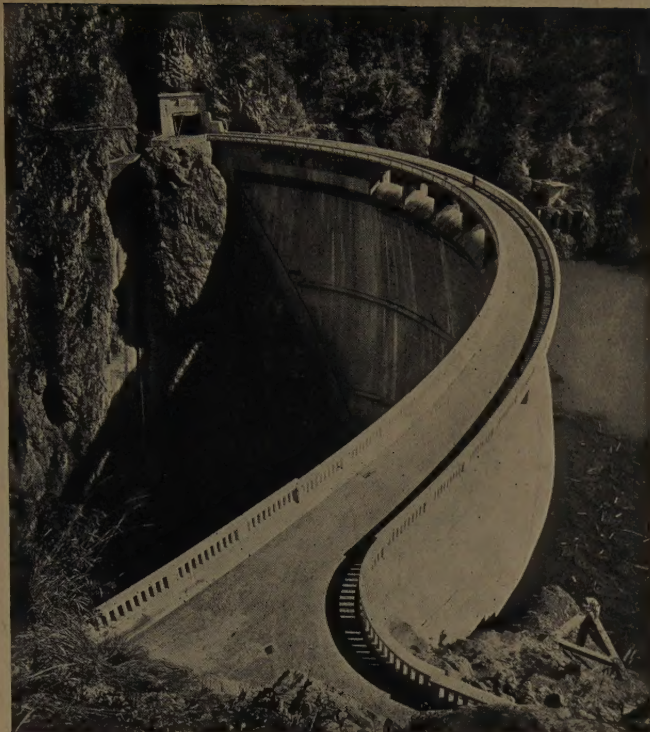
The tests on normal 1-to-3 mortar gave the results shown in Table 1.

TABLE 1

	After 3 days	After 7 days	After 28 days
Tensile strength :			
kg. per sq. cm.	31·3	36·1	45·0
lb. per sq. inch	445	513	640
Compressive strength :			
kg. per sq. cm.	414·0	555·5	709·5
lb. per sq. inch	5,710	7,900	9,970

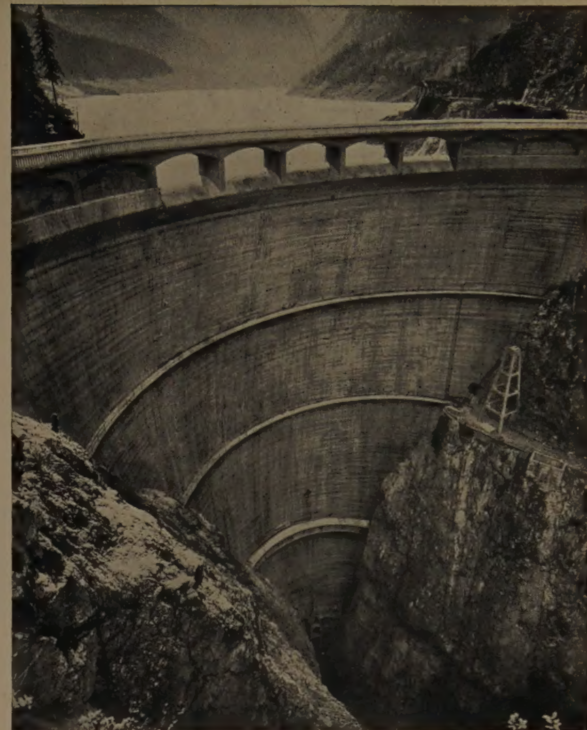
Setting started after 3 hours
Setting completed after 5 hours 15 minutes.
Crushing fineness, 900 mesh : 0·2 per cent
" " 4,900 " 1·0 per cent

Fig. 4



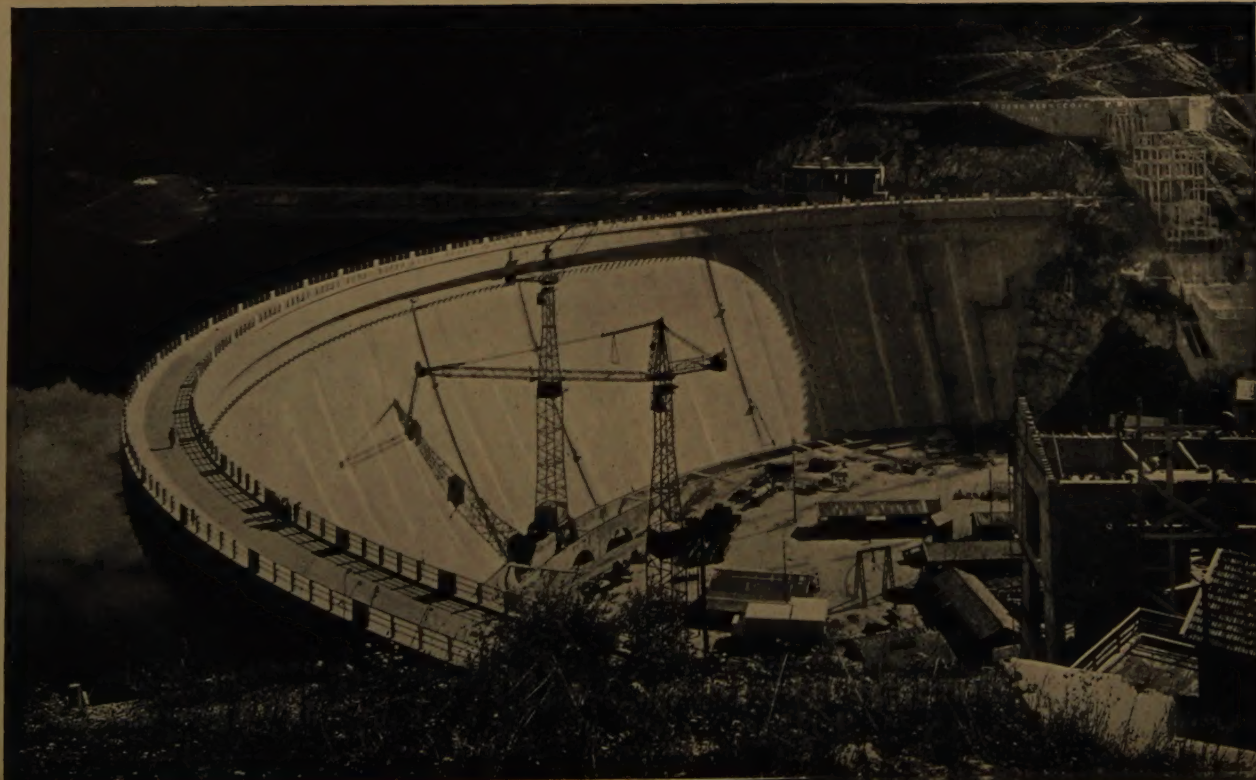
The Lumiei Dam. View from Left Bank

Fig. 5



The Lumiei Dam. View from Downstream

Figs 11



The Pieve di Cadore Dam. Downstream Face

Fig. 15

The Val Gallina Dam. Concreting in Progress

Fig. 19

The Valle di Cadore Dam. Upstream View

The composition of the concrete is shown in Table 2.

The average strengths of the concrete made during the years 1946 and 1947 are shown in Table 3.

Several tests were carried out on cubes of concrete extracted from the body of the dam itself during its construction. The tests showed the concrete to have a compressive strength of 528–691 kilograms per square centimetre and to be completely impermeable.

TABLE 2

		Kg./cu. m.	Lb. per cu. m.*	Lb. per cu. yd.†	Per cent
Ferro-pozzalanic cement		270·0	595	455	10·8
Very fine sand		30·0	66	51	1·2
Sand	0–4 mm. (0–$\frac{5}{32}$ inch)	562·5	1,240	948	22·5
Aggregate	4–10 mm. ($\frac{5}{32}$–$\frac{3}{8}$ inch)	187·5	413	316	7·5
	10–40 mm. ($\frac{3}{8}$–$1\frac{9}{16}$ inch)	700·0	1,543	1,180	28·0
	40–80 mm. ($1\frac{9}{16}$–$3\frac{5}{32}$ inch)	750·0	1,653	1,265	30·0
		2,500·0	5,510	4,215	100·0

Water content: 144 litres per cubic metre

Water/cement ratio: $\frac{144}{270} = 0{\cdot}53$

The differences between the strengths so measured and those obtained in the laboratory tests were probably due to the use of the vibration technique when pouring the concrete for the body of the dam.

The temperature rise in the body of the dam during setting was measured with electric thermometers. The maximum temperature-rise recorded was 31° C.

The pozzolana was brought from near Rome by special trains to the cement works at Udine. The cement was carried from the cement works to the site in cylindrical containers of about 8 cwt capacity, following a system that had already been used in Italy for the Fraele dam in Valtellina and in Switzerland at St. Barthèlemy. The containers arrived by rail at the station of Villa Santina and were conveyed to the dam on a cableway 18 kilometres long.

TABLE 3

		28 days	90 days
1946	Kilograms per square cm.	326	384
	Lb. per square inch	4,717	5,462
1947	Kilograms per square cm.	353·8	409·7
	Lb. per square inch	5,032	5,827

* Incorrectly printed in advance proof as "Lb. per cu. yd."
† This column added after discussion (see p. 543).

The empty tanks, after the automatic discharge of the cement into four silos, each of 400 tons capacity, returned by the same cableway. The construction work was carried on without interruption during the war year of 1943 and the post-war years 1946 and 1947, but that would not have been possible without the collaboration of everyone interested in the project—the Ministry of Public Works Dam Service, the production factories, and especially the State Railways. The transport difficulties were, however, sometimes quite disheartening.

The water used for washing and mixing was drawn from a tributary on the left of the Lumiei. This water, unlike that of the Lumiei itself, did not contain any sulphates.

The aggregates were obtained from a funnel-shaped quarry opened on the left bank, high above the dam. The rock was the same as that of the abutments—a limestone of very good quality. It was excavated from the sides of the funnel and discharged into a vertical shaft, at the base of which it was collected on a wooden platform. From this platform it fell through suitable openings into small trucks and was carried to the crushing and concrete-mixing plants.

These were installed just downstream of the dam at a height of 50 metres. Two intermediate bucket-elevators increased the available fall to 73 metres.

The material was discharged from the trucks into a crusher, from which it passed to a cylindrical revolving screen with an aperture size of 88 mm. The stone larger than 88 mm. was then ground in three crushers and, together with the material issuing from the revolving screen, was carried by an elevator to a rotary screen with an aperture-size of 40 mm. This screen sorted out the 40–80-mm. stone and poured it directly into a silo.

The under-40-mm.-fraction was carried to a vibrating screen with an aperture-size of 10 mm. The retained fraction was discharged directly into another silo, whilst the screenings (0·1 to 10 mm.) were carried to gyratory crushers and to a Magutt 66 hammer-mill. (These two machines could also deal with stone larger than 40 mm., which could be taken directly to the chutes leading to the appropriate silo.)

Then followed a vibrating screen, with an aperture size of 10 mm., which discharged its retained fraction to a Symons-type rotary screen and the two screenings were carried by an elevator to a further vibrating screen, with an aperture-size of 12 by 4 mm., the products being conveyed to their corresponding silos.

The product of the Magutt hammer-mill could be conveyed directly to the 0–44 mm. silo or to the lower elevator carrying it to a vibrating screen with an aperture width of 4 mm.

The 0–4-mm. screenings from this screen were reduced to dust by means of a ball-crusher.

Four volumetric batchers installed below the silos supplied two 1-cubic-metre concrete-mixers. In the same manner as for the cement,

the very fine sand was weigh-batched. The water was also automatically measured. Only one workman was required to operate all the weighing machines, concrete-mixers, and batchers.

From the mixers the concrete was then hopper-loaded into bottom-dump buckets and taken by trucks or blondins, to the hooks of four derricks. It was placed in alternate bays and vibrated.

The maximum volume of concrete poured daily exceeded 600 cubic metres. The horizontal section of the dam at the different levels was reasonably constant.

Special steel forms of small size (0·5 by 0·5 metre) were used for shuttering. They proved to be suitable for the double curvature of the two surfaces.

The two concrete-placing derricks on the left side were served directly from the concrete-mixers by bottom-dump buckets on trucks and the two on the right were served by cableways, which carried the buckets from one side of the gorge to the other. In that way any segregation of the concrete was avoided. When the work was finished, the joints were cement-grouted, in order to ensure the continuity of the compressive strains in the structure under load.

A deep grouting curtain was made in the rock along the abutments and on the bottom. The main diaphragm of this curtain was built up by means of a series of injections carried out every 5 metres downstream from the dam, normal to the sides of the gorge and intersecting the largest possible number of surfaces separating the rock-layers; another diaphragm sealed the saddle to the rock. For the second diaphragm, grout injections were made every 2 metres on the right side and every $2\frac{1}{2}$ metres on the left. The grout holes were drilled through the saddle from the upstream side; they crossed the area of contact of the concrete with the rock and intersected the main diaphragm. Altogether 2,306 holes, having a total length of 19,262 metres, were drilled, and 15,226 quintals of cement was injected.

The following control and measuring apparatuses (shown in Fig. 3, Plate 1) were installed in the dam :—

- 32 electric thermometers embedded in the concrete;
- 4 ,, ,, in the open air;
- 3 ,, ,, submerged in the water;
- 18 groups to measure the strains, each composed of 3 electro-acoustic extensometers (1 vertical, 1 horizontal, and 1 isolated) embedded in the concrete near the upstream surface;
- 62 seats for strain gauges placed on the downstream surface—horizontal, vertical, and inclined at 45 degrees;
- 9 seats for clinometers on the downstream surface;
- 2 observation places with collimator field-glasses; and a 3-sight range fixed on the dam.

In addition, local triangulation and high-precision levelling networks

were established in order to detect eventual movements of the dam and the banks.

The construction * of the dam was started in the winter of 1941–1942, with the diversion tunnel. The excavations for the abutments required nearly two whole working seasons and only at the end of the autumn of 1943, when the Nazi occupation had begun, was the pouring of concrete commenced. Shortly afterwards the work had to be stopped, and it was recommenced only at the end of 1945 to complete the concrete-mixing plant. Full pouring started in April 1946 and the dam was completed in November 1947.

The excavations were made in the shape of funnels and the debris was tipped down chutes previously dug inside the rock that had to be excavated for the abutments. From the bottom of the chutes the debris was carried, through small horizontal tunnels, to the river bed upstream and downstream of the dam. With this system it was possible to start the excavations at different elevations simultaneously, and to prevent the debris from falling into the gorge on the site of the dam itself, whence it would have had to be removed again. This system proved very efficient in practice ; it was used also for the steepest sides of the gorges of the Piave and Val Gallina dams and it will be adopted for the Vajont dam. The problem of excavations for such dams of considerable height, located in gorges with very steep sides, is of great importance, since this work takes as much time as the pouring of the concrete itself.

Three discharge outlets, placed at different heights, are connected to the body of the dam through the left-hand abutment. Their typical design shows that the section of the tunnel narrows from a 4·2-metre-diameter circle to a 2·40-by-1·60-metre rectangle, and is closed by two interconnected slide-gates, which are controlled by oil-pressure supplied by a general reservoir.

The diversion-tunnel has been blocked by a concrete plug into which is set a steel pipe of 1·20 metre diameter. The open side of the pipe is closed by a steel plate which, in an emergency, could be destroyed by explosives.

Near the discharge-tunnel, at a level of + 890·00, and in communication with the adit to the gate chamber, an auxiliary generator was installed and connected to a Pelton wheel which starts to work automatically when the pressure of the oil for the control of the slide-gates falls below a certain value. Thus, the control of the slide-gates is ensured, even in the event of power interruptions.

The dam has been in service for more than 3 years, during which time it has been subjected to extremes of load-conditions, including maximum water level and a serious earthquake. Its behaviour has been entirely satisfactory and no inconvenience worth mentioning has occurred. A few insignificant leakages appeared at the beginning, but they decreased

* The contract was placed with the Italian firm Giuseppe Torno and C.S.A., of Milan, who also built the Piave dam.

progressively and have now almost completely disappeared. This is thought to be the result of both the grouting and the chemical action of the pozzolana, which, by the gradual fixation of the free calcium-oxide residue in the concrete after the setting, progressively eliminates water leakage.

A General Outline of the Piave-Boite-Vajont Scheme

The dam on the River Piave at Pieve di Cadore is one of the essential components of the Piave-Boite-Vajont scheme provided for the utilization of the river and its tributaries upstream from the Piave-Santa Croce scheme. The latter, completed in 1929, was carried out for the purpose of regulating the flow of the river at Soverzene (near Ponte nelle Alpi), by means of Lake Santa Croce (a seasonal reservoir with a net capacity of 121 million cubic metres), and conveying its water as far as the Venetian plain along the ancient glacial furrow of the Piave towards Vittorio Veneto and the river Livenza.

Table 4.—Capacities of lakes and reservoirs shown in *Fig. 6* (p. 509)

Lake or Reservoir	Cubic metres	Cubic feet	Gallons
Campocroce	4,830,000	169,560,000	1,061,320,000
Comelico	1,208,000	42,400,000	265,330,000
Forno di Zoldo	31,692,000	1,112,400,000	6,962,800,000
Lago Morto	3,015,000	105,840,000	662,480,000
Pieve di Cadore	64,700,000	2,270,700,000	14,212,900,000
Podestagno	39,138,000	1,373,760,000	8,598,720,000
Romotoi	5,530,000	194,130,000	1,215,110,000
Sappada	11,080,000	388,800,000	2,433,600,000
S. Caterina	6,640,000	233,010,000	1,458,470,000
S. Croce	120,770,000	4,240,000,000	26,533,000,000
Vajont	60,400,000	2,119,500,000	13,266,500,000
Valle di Cadore	3,823,000	134,190,000	839,930,000
Val Gallina	5,840,000	204,930,000	1,282,710,000
Val Visdende	17,107,000	600,480,000	3,758,560,000
Vodo	704,000	24,705,000	154,635,000

This route is much shorter than the one along the actual course of the Piave itself and passes through Lake Santa Croce and other little reservoirs —either natural or easily adapted from small lakes. Further, it allowed the water to be led to a much lower outlet-level—13 metres above mean sea level—whilst at Ponte della Priula, the outlet into the plain of the course of the River Piave is 70 metres above mean sea level.

This big scheme, which was completed in 1929, has almost divided the drainage area of the Piave, from a hydraulic point of view, into two parts, each having approximately the same catchment area but not the same average elevation above sea level; the first, which may be called the north-eastern area, lies north of Soverzene and its waters are mostly conveyed to Lake Santa Croce and to the Piave-Santa Croce scheme; the

second, which is downstream from Soverzene, is quite independent of the first. The S.A.D.E. has planned, and will soon carry out, the utilization of the water from both areas, according to comprehensive schemes.

The present output of the Piave-Santa Croce plant is 600 million units evenly distributed throughout the year. The plant can easily convey a larger average flow than the one foreseen in the designs. But, owing to the lack of upstream control, and since the capacity of Lake Santa Croce cannot be proportionally increased, a larger output would not be possible apart from negligible seasonal increases. For this reason the development of plants upstream from Soverzene was studied with a view both to producing energy in the new stations and to increasing the average output of the Piave-Santa Croce plants by water-storage upstream.

A long investigation was made of the previous projects which had been conceived independently of one another, and consideration was given to the necessity of preserving another power plant already existing upstream (the Piave-Ansiei scheme with the tailwater at Pelos at a level of + 683·50 metres—and an output of 120 million units). The solution finally adopted was a single plant connecting the Piave and its tributaries Boite and Vajont between these levels, and the intake of the Piave–Santa Croce plant situated at a level of + 390·50 metres, the fall being 293 metres.

The following is a general outline of the scheme. A reservoir with a net capacity of more than 64 million cubic metres was built on the Piave. A pressure-tunnel, 24 kilometres long, with a diameter of 4·5 metres, conveys the water of the Piave reservoir to two other reservoirs placed in series; the first is 18 kilometres downstream in the Vajont valley, a left tributary of the Piave, with a net capacity of 58 million cubic metres; the second is at a distance of 24 kilometres in the Val Gallina, and has a net capacity of 6 million cubic metres. The water of the River Boite, diverted by means of a small arch-dam, will join the water coming from the Piave through a tunnel 4 kilometres long reaching the main tunnel 3 kilometres downstream from the Piave intake works. From the Val Gallina, two parallel tunnels, 2·5 kilometres long, with a diameter of 5 metres, will convey the water to the big power station, at Soverzene, which is equipped with four vertical sets of Francis turbines and generators, each having a capacity of 55,000 kilowatts.

The annual output of the new plant will be 650 million units and an additional annual output, amounting to 150 million units, will be obtained automatically by increasing the average flow through the Piave-Santa Croce plant; in this way, the north-eastern system of the Piave has now been made complete from a level of + 830·00 down to + 13·00. Previously the system, including the Piave-Santa Croce and the Piave-Ansiei, had produced 720 million units; it will now produce 1,520 million units, evenly distributed throughout the year. This has been achieved by the introduction of only one power station and by increasing the total net capacity of the system of reservoirs to more than 250 million cubic metres.

Further extensions of this system are foreseen ; these include the deviation of the river Maè, a right tributary of the Piave, into the Vajont reservoir, and the building of a reservoir on the Maè itself.

The new output will therefore reach nearly 1,000 million units and will be further increased by the completion of other schemes on the upper Piave and the Boite.

The development of the projects upstream from Soverzene, will, of course, make the subdivision of the Piave drainage area into two parts progressively more complete.

The topography of the area and the design adopted for the scheme made it possible to plan the construction of the Piave-Boite-Vajont plant in different stages, and there were accompanying financial and practical advantages.

The first stage, which was completed in 1950, includes the Piave dam, the tunnel from the reservoir of the Piave to the Soverzene power station, only one of the two tunnels from Val Gallina to the power station, and two generating sets in the power station with an output of about 500 million units. Owing to the installation of a subsidiary power station at Perarolo, which has been in operation since January 1948, the annual output has been raised to about 100 million units. The second stage, including the dam and tunnel for the deviation of the Boite, the dam of the Val Gallina, the second tunnel from the reservoir to the power station, and the third set, was completed by the end of 1951.

The last stage, including the Vajont dam and the fourth set will be started later ; it will take at least 4 years.

The determinations of many elements of the project, chiefly the diameters of the tunnels and the maximum intake levels of the Vajont and Val Gallina reservoirs, was the result of many long analyses, based on flow statistics covering a period of 25 years.

The inclusion of the Vajont and Val Gallina reservoirs, even without considering their function as flow-regulators, led to considerable saving in the proportions of the tunnels, which, as far as Val Gallina, are not large enough to carry the peak flow, although they can cope with high average flow. There was also a substantial saving in the surge tanks, which have to face the oscillations of the mass of water from a tunnel 2·5 kilometres long, instead of one 27 kilometres long.

The Dam on the River Piave

See Figs 7 and 8, Plate 2, and *Figs 11* (between pp. 512, 513)

After a long and expensive geological research, based on core-drillings and exploration tunnels, in many sites in the valley between Pelos and Perarolo, a location was chosen which was not entirely a favourable one, but is actually the most suitable from both topographical and geological points

of view. The site is cut out in the Dolomitic limestone (Upper Trias) in the form of a large trapezium, about 300 metres wide at the base and about 55 metres high, formed by a rocky plateau (Pian delle Ere) flattening out on the left. On the right, the plateau is cut by a narrow gorge, 55 metres deep, its direction converging with the axis of the valley. The investigation proved that the left plateau was formed wholly by solid rock, with the exclusion of any ancient geological gorge. Owing to the shape of the section, consideration was first given to blocking up the gorge on the right with a thin arch and completing its left abutment upon the plateau by a big gravity structure ; from this point to the left, the barrage would have been completed by a solid or hollow gravity dam. However, model-tests made from 1939 onwards proved that tension stresses at the base of the gravity structure were very high. In order to eliminate them, it would have been necessary to increase its volume to such an extent as to make this combined solution too expensive.

The simplest solution would have been a straight gravity dam supported by the plateau and a plug to close the gorge. There were many objections, however, chief of which were the width required for the toe of the plug (and consequently its volume) and the uncertain static conditions arising at the joint between gravity dam and plug. These, together with many other factors, especially the direction of the gorge and the gradual downstream divergence, led to the idea of using an arched form for the upper structure. However, the adoption of an arch-gravity dam required very thorough investigations to be carried out, for there were many unknown factors, principally the following :—

1. How much reliance could be placed upon the behaviour of a gravity arch of such dimensions.
2. The stress conditions at the transition from arch to plug.
3. The problem of whether the plug would act as a wedge or a thick arch.

It was therefore considered necessary to combine mathematical analyses with thorough model-tests. After a systematic series of calculations and experimental studies, the structure assumed the following features. The upper arch has an average radius of 160·90 metres, a length of 390 metres, a chord of about 305 metres, and an average height of about 55 metres ; so that there is a chord/height ratio of 5·5, which is slightly higher than that of the Gibson dam. The term " average radius " is used because the top arch of the dam is polycentric (with slight deviations from the circular), whilst the one at the base is actually circular. That is to say, the vertical sections change from one point to another of the arch, to make the arches approach as nearly as possible the curve of pressures. The vertical sections have a pronounced curvature upstream with a slight overhang. The thickness at the base is 26 metres, which is about 65 per cent of the thickness of a solid gravity dam of the same height.

Towards the abutments the thickness increases by about 20 per cent. The plug is 53 metres long and its thickness on the axis of the gorge decreases from 36 metres at its base to 26 metres at the top (matching the wall of the dam).

The total volume of the dam is 377,000 cubic metres, which is at least 120,000 cubic metres less than the volume of any other types; this corresponds to an actual saving of about 1,000 million Italian lire (based on costs obtaining in 1949).

Another advantage of this solution was that, along its radial sections, the distance from the toe of the arched dam to the edge of the plateau towards the deep gorge of the Piave was almost constant. It may be said, therefore, that the ground elevations of the single cantilevers—except of course those in the plug or in its immediate vicinity—did not substantially differ from one another. The mathematical analyses were carried out mainly by Professor Filippo Arredi of the Faculty of Engineering at the University of Rome, who, starting with the studies of Smith, Conti, and Tölke, followed mostly original methods of calculation. Other interesting studies, based on Tölke's method, were made by Professor Guido Oberti of the Polytechnic School of Milan, with Professor Arturo Danusso as a consultant. Their results were then checked by both theoretical and experimental methods, based on the deformation of elementary two-dimension photoelastic models of cantilevers and arches. From these deformations, radial deflexions were obtained to be included in the system of equations, solving the problem of the imaginary network of arches and cantilevers.

In addition, the Trial Load method of the American Bureau of Reclamation was carried out by Professor D. Tonini of the Author's department; this method is, however, very laborious. Finally, static behaviour of the whole structure has been thoroughly checked on a $\frac{1}{40}$-scale model, for which a special experimental basin was built at Bergamo.

This basin forms the nucleus of a laboratory which is being enlarged and will contain other appropriate installations for model-tests of large structures.

The topographical character of the rocks was imitated by the preparation of a similar foundation-bed of prisms. The model was cut horizontally by radial joints into single blocks, which corresponded to the actual structure. The concrete of the model was prepared so as to reach a proportion of 2 : 3 between its own modulus of elasticity and that of the foundation, bearing in mind the mechanical similarity. The joints which had been left open were then closed with mortar similar to the one used for the model.

A special apparatus reproduced the effects of both dead weight and water pressure, by means of a system of hydraulic jacks, whose forces acted on the model through a system of unbalanced levers and transmission straps. The model was divided into 272 elements and the individual

forces were applied at the centre of gravity of each element by means of independent conduits and distributing slabs.

The hydraulic pressure on each slab corresponded to a rectangular section a little larger than that of the slab. By this means, it was possible to have, at the margins of the model, vertical and horizontal spaces in which to place the measuring apparatuses and to facilitate control of the water-side surface.

The measured movements of the model were of the order of 0·01 mm. To determine the absolute differences, the instruments were placed on a steel frame entirely independent of the model.

The local movements were measured on the air-side surface, on the crest arch, and on the places of the water-side surface which could be easily reached, by means of instruments with a high amplification-ratio.

For the parts of the water-side surface which were accessible only with difficulty, electro-acoustic strain gauges were installed. The local alterations could be controlled with a precision of 0·01 mm. Tilting of cantilevers was measured by clinometers with a precision of $\frac{1}{60}$-second of arc.

In consequence of the studies made by Professor Arredi in 1948 and of further model-researches, the shape of the dam was slightly modified, and the original model was broken into many parts and rebuilt according to the definite designs. Like the preceding model, it was divided by vertical radial joints and cemented together by injection when the process of shrinkage was completed.

The researches made on the second model were complete and reliable and made possible the precise establishment of the definite form of the structure.

In fact, the remarkably close agreement of the results of analytical and model researches has been completely convincing. It may be added that the model tests, stressed much above the actual maximum, showed that the structure behaved perfectly and did not show the slightest crack. Shear stresses were practically non-existent. Furthermore, the percentage of the load absorbed by the arches proved to be much greater than we expected, especially because the yielding of the rock, considered as a whole with its fissures and discontinuities, turned out to be remarkable. The rock has a modulus * of elasticity much inferior to that of the concrete and the yielding of foundations is therefore considerable. The calculations were based on a modulus of 100,000 kilograms per square centimetre.

Another noteworthy conclusion drawn from the model-tests was that the structure did not conform with the theory of horizontal arches. It behaved as a set of arches lying in inclined planes, with their central parts high in the middle and their abutments low on the sides. Such behaviour of the structure is essential in triangular- or trapezoidal-shaped locations; under such conditions the lateral parts of the crest have relatively low stresses.

* The range of values is given in the MS.

Model-tests were also carried out for the plug, because of the difficulty of submitting it to any theoretical analysis having even a tolerable resemblance to its actual static behaviour. The peculiar shape chosen (aptly called "bean-like") resulted from the elimination of all the concrete parts which did not enter into the static behaviour of the structure, inside which thin internal arches, with small radii (known as "Rèsal arches"), tended to work.

From the horizontal sections at different heights, it appears that the arches widen upwards to link themselves to the structure above, in accordance with a general principle of continuity to which the Author gives the greatest importance; that is to say, a continuity which ensures the smooth absorption, transmission, and distribution of the stresses between the various parts of the structure and between the structure and the foundation soil. Therefore, wherever possible, sudden changes of level and inclination in the contact-surface between the different elements of structure and foundation are avoided. Even greater importance is attached to continuity than to symmetry.

Absolute continuity of the structure during construction is not implied, because construction joints are obviously necessary.

In the case of the design of the Piave dam, it should be noted that, owing to the type of structure, no uplift pressure was taken into consideration as a force acting on the dam and, to ensure its elimination, an adequate network of drainage pipes was provided, together with a series of tunnels for inspection and collection of water.

The abutments are curved and have perimeter joints similar to those of the Lumiei dam. The perimeter joint at the base is also curved, being almost normal to the line of pressure, curving with a general slope upstream. It runs horizontally, following the abutments, on both sides of the valley, at a level of + 634·00 metres on the upstream side and at a level of + 635·82 metres on the downstream side. The dam is cut by vertical joints into 33 blocks, each about 12 metres long, along the axis of the dam. All the joints are sealed close to their upstream faces by a staunching piece. The two abutments are practically symmetrical.

The work carried out during 1948* included the placing of concrete for the plug and the whole base of the dam up to the surface of the perimeter joint. That was the stage which it had been planned to reach before the expected seasonal interruption of work. Only the last two blocks of the footings of the left abutment were not completed, owing to the approach of winter, but that part of the work was completed immediately at the beginning of the 1949 working season. The placing of the concrete for the corresponding upper part of the arch followed later, so that the whole arch gravity structure lay upon concrete that had appreciably matured.

The concrete was placed in lifts 1 metre thick. Since the results of

* See footnote * on p. 516.

mathematical analyses confirmed the importance of temperature phenomena, that was the greatest problem. The first step taken towards the solution of the problem was the adoption of a suitable cement. After long researches, resort was made to a low-heat type of ferro-pozzolanic cement, containing an addition of 25 per cent of Roman Pozzolana, with a tri-calcium-silicate content of not more than 40 to 45 per cent. About 1 per cent of "Plastiment" was added. The results were exceptional; as demonstrated by the diagrams of temperature inside the dam, there was a maximum increase of temperature of no more than 15° C., in comparison with the 35° to 30° C. rise usually encountered in dam construction.

In the first two months (March and April) and the last two months (October and November) of the concreting season a slightly different sort of cement (called "autumn cement") was used; it had a rather higher proportion of tricalcium silicate and pozzolana, to ensure more rapid setting of the concrete and to allow a corresponding increase in the daily output.

The concrete had a cement-content of 200 kilograms per cubic metre. The part upstream, with a variable thickness from 1·50 metre at the crest to 4·00 metres at the base, had a content of 250 kilograms per cubic metre; the average for the whole dam was about 220 kilograms per cubic metre.

The addition of Plastiment, by allowing a reduction in the water-cement ratio, improved both the strength of the concrete and the bonding between the lifts. (The latter had been previously demonstrated by Bolomey.) It has the slight disadvantage, however, of delaying the setting.

Tests under maximum hydrostatic load showed the concrete to be highly impermeable.

The concrete mixes used for face and core are shown in Table 5.

TABLE 5

	Face-concrete	Core-concrete
Cement	9·5 per cent	7·6 per cent
Sand 0–4 mm.	35·5 „ „	26·4 „ „
Coarse sand 4–16 mm.	23·0 „ „	20·0 „ „
Aggregate 16–40 mm.	16·0 „ „	17·0 „ „
Aggregate 40–75 mm.	16·0 „ „	15·0 „ „
Aggregate 75–120 mm.	—	14·0 „ „
	100·0 „ „	100·0 „ „
Approx. water/cement ratio	0·5 „ „	0·55 „ „

The cement used during the 1948 concreting season was so good that it became possible to rescind the earlier decision to build the arch with open vertical joints of a width of 1 metre (as had been done at Rossens where the system had been unavoidable owing to the characteristics of the

cement). The decision was also influenced by the consideration that the differences of temperature, which rapidly arise between the external surface of the blocks and the zone immediately behind it, would have produced very undesirable internal longitudinal stresses. Another procedure adopted, to contend with temperature phenomena, was to reduce tractional forces at the joints between the blocks by means of a partial coating with a bituminous substance near the faces.

The preliminary works (temporary dam and diversion tunnels) had been started in 1942/43. Work was resumed in 1945–46 with the excavations, and at the end of 1946 the contract was let to the same firm of contractors who built the Lumiei dam.

In 1947, the construction plant was completed; in 1948, the pouring of concrete was started, and 135,000 cubic metres placed; pouring ended on November 3rd, 1949.

The construction plant was very large; the situation offered by the rocky plateau, levelled to working plane by excavated debris, was exceptionally good for plant installation and full advantage was taken of it. The plant for concrete mixing had been designed for a maximum output of 120 cubic metres per hour. In September 1948, a maximum of 27,000 cubic metres per month was attained, and in the summer months of 1949 a satisfactory regular output up to a maximum of 40,000 cubic metres per month was reached.

The work had been divided into two shifts and, in addition, the whole surface of the arch was available, whilst in 1948 the surface was practically limited to the plug, because the foundations on the plateau have only a limited thickness.

The cement was brought, in the usual containers, on railway trucks from the factory at Vittorio Veneto to the Sottocastello station which is quite near the plant. From there, it was carried by Fuller pumps to big silos in the middle of the plateau, and thence to the mixer-silos by other Fuller pumps.

Aggregates were obtained from a funnel-shaped quarry in dolomitic limestone about 600 metres from the plant, to which it was linked, for more economical and speedy transport, by two shafts and a tunnel.

The maximum grading of the aggregate was 150 mm. The concrete was vibrated by Notz Super vibrators, with an average frequency of 11,000 vibrations per minute, the results of which are excellent both in regard to strength of concrete and to imperviousness; the samples taken from the body of the dam and from test drillings made in the plug showed most satisfactory results. The density of concrete 7 days old was more than 2,600 kilograms per cubic metre.

The accessory works are the upstream cofferdam, which is a remarkably small thin arch, 31 metres high, with a maximum thickness of 2 metres; the bottom and middle discharge tunnels, excavated in 1942, which are to be used as diversion tunnels, below the rocky plateau of Pian delle Ere;

and a spillway situated on the left. The total discharge capacity is 1,100 cumecs. In the event of an exceptional flood, overflowing through the openings in the parapet may be safely permitted, aeration being assured by the presence of reinforced-concrete pillars on the parapet itself.

The intake works are also on the left; they consist of two parallel tunnels each 3·5 metres in diameter, fitted with screens and rakes and double gates.

The general results of the construction are excellent. From the point of view of imperviousness, only in some areas on the left shoulder, with the maximum hydrostatic load, has there been a leakage of a few litres per second. This is negligible, since the nature of the limestone is not affected by the presence of water. However, to reduce this leakage, supplementary holes and grouting work was carried out on the left shoulder. By the end of September 1950, a total length of 62,000 metres had been drilled and 5,700 tons of cement injected. The injections have been carried to a depth of more than 135 metres.

The figures reported above include, first, the vertical water-proofing screen upstream, and secondly some special work carried out on some rather badly cracked areas of the Pian delle Ere, which might be called a process of homogenization by grouting. The aim of this healing action, besides improving the water-tightness, was to ensure a gradual and uniform distribution of the stresses from the foundation to the rock, without abrupt variations in the flow.

The grouting of the arch-joints was completed in March 1950, before the maximum level of the reservoir had been reached.

To check the working of the dam, the following instruments were installed in its structure (see Fig. 9, Plate 3, and *Figs 10*) :—

To measure deformations :
- 103 Galileo electro-acoustic strain gauges ;
- 57 Carlson electric strain gauges ;
- 17 Carlson tensiometers.

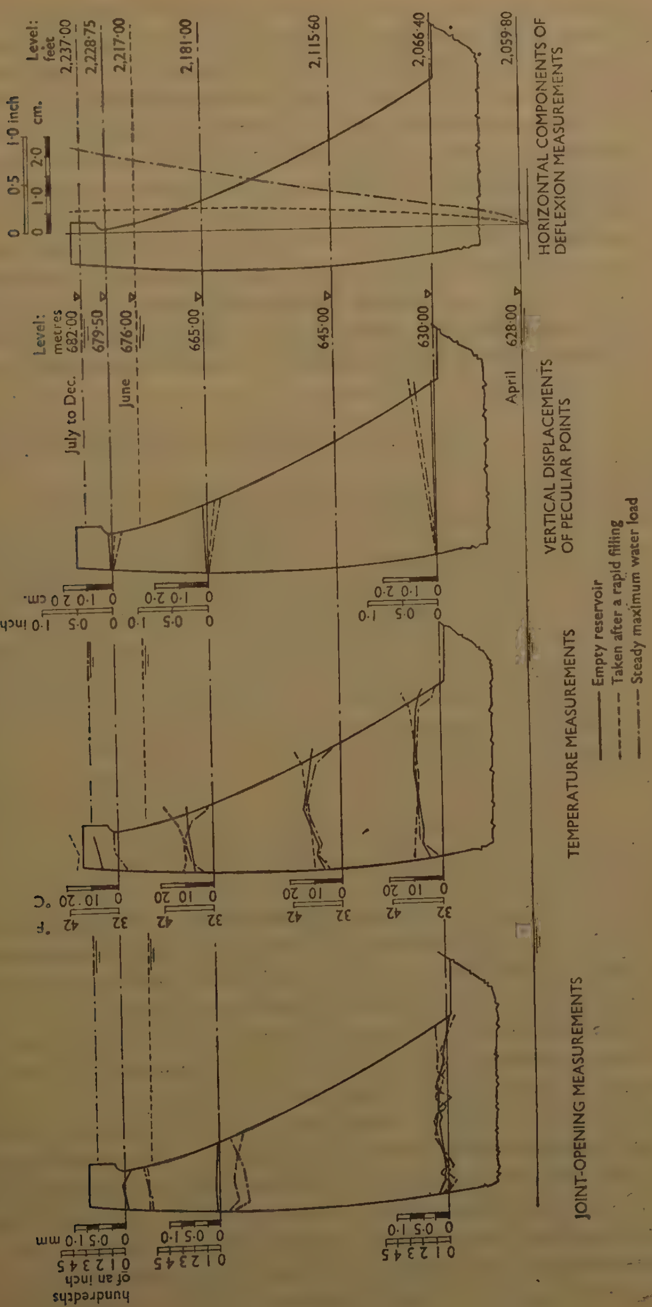
To measure temperatures :
- 87 Siemens electro-thermic gauges (71 in the concrete, 7 in the water, 9 in the air).

To measure the humidity of the concrete :
- 66 Brazey teleigrometers.

The movements of the dam and the rocks on the sides of the valley were checked by two networks—one of high-precision levelling, and one of triangulation. The levelling network was extended to about ½ kilometre downstream and upstream of the dam to determine the extent of the influence of the reservoir, and the following information is being obtained :

The movements of the dam top, by means of a special fixed-base 80-magnifications collimator, allowing readings at a distance of more than 500 metres.

Figs 10



THE PIEVE DI CADORE DAM. SUMMARY OF THE MEASUREMENTS CARRIED OUT IN 1950

The movements of the characteristic points of three cross-sections, by means of Huggenberger co-ordimeters (seven), and eleven positions for the Galileo microtelemeters with the method of the plumb rule.

The plane movements in 117 positions established in eight sections by means of Galileo elongated-base clinometers and of four pairs of continuous photographic registration clinographs.

The joint-movements in 103 pre-selected points, by means of Galileo deformometers.

The movements between the structure and foundation in three shafts 5 metres deep under the foundation, by means of three Galileo slide-gauges (seat combination for the deformometer and clinograph).

Any possible local land movements, by means of three seismographs. (Owing to the seismic nature of the area it is important to identify the causes of such movement).

Up to the present, most of the instruments have been working regularly and satisfactorily.

In the next few years, the information obtained will result in a unique collection of experimental data on the behaviour of large-arch gravity dams, and may well allow economies to be made in future works of a similar nature. A very careful investigation will have to be made, because the safety of entire valleys is involved. However, it seems obvious that, when the results of tests on the model and of experiments on similar structures already built agree satisfactorily, it should be possible to reduce substantially the safety coefficients now in force for this type of dam, and at the same time keep them at a materially higher level than those in force for less important structures.

The Val Gallina Dam

See Figs 12, 13, and 14, Plate 4, and *Fig. 15* (facing p. 513)

This dam represents another step forward in the construction of dome-type dams. It was constructed only after exhaustive investigations including drilling, tunnelling and grouting tests, since the rock (Upper Trias limestone) appeared to be rather cracked on the surface, and the possibilities seemed doubtful. The original cross-section of the site was far from being symmetrical, and symmetry was obtained by blocking the deepest part of the gorge with a plug.

The longitudinal section is shell-shaped, and the vertical section dome-shaped with a pronounced overhang downstream. The height is 85 metres and the volume of concrete 98,000 cubic metres; the chord/height ratios are 2·7 : 1 inside the perimeter joints and 2·15 : 1 for the whole construction including the abutments and closing plug; these figures are rather high for a thin arch dam. Steel reinforcement is provided in both water-side and air-side faces. Owing to the shell-shape, a considerable part of

he load is absorbed by the cantilevers. Important information on the ctual working features of this type of construction has been obtained from nodel-tests designed and directed by Professor Guido Oberti.

The reservoir of Val Gallina, with a capacity of 5·9 million cubic metres erves as a storage-and-ponding reservoir for the water-flow to the Sover- ene power station on the River Piave. It also acts as a surge-tank and hus assists control of the turbines. Owing to its ponding effect, it was ossible to avoid the construction of a second conduit (5·6 kilometres ong) from the Vajont reservoir to Val Gallina, and also to reduce the pro- portions of the surge-tanks of the Soverzene power-station. The conse- quent economy completely justified the construction of the dam.

The project for the water-intake presented some difficulties, since only one tunnel enters the reservoir of Val Gallina, while two parallel tunnels leave it. Moreover, a by-pass tunnel has been built which has to be con- nected either with both tunnels at the same time, or with either of them separately. Consequently, several gates will be required.

The cement content of the concrete mix was 250 kilograms per cubic metre, the cement being the same ferro-pozzolanic cement as used for the Piave dam during the cold months.

The grouting operations have assumed considerable proportions, and a thorough series of cement injections were made. Up to March 31st, 1951, a total length of 32,201 metres had been drilled and 25,085 tons of cement injected, for the curtain and the homogenization.

Gravel and sand was obtained from the heaps of debris gathered by the confluence of the Gallina and the Piave.

For the excavations, three pairs of tractor-drawn scrapers were used each having a capacity of 3 cubic metres, which poured the material into a cement-hopper, under which a screen of bars was arranged. The round stones left behind were sent through a Ross-system conveying plant, to a type-500 Cleman crusher. Two elevators, 21 metres high, carried the material from the crusher, together with the material from the screen, up to two washing screens which extracted the stones over 40 mm. in size. The stones under 40 mm. in size were passed through two vibrating screens which further separated the sizes—10–40 mm., 4–10 mm., and under 4 mm. Four conveyor belts carried the separate components to four vertical silos; the 4–10 mm. stones were crushed by a mill, situated by a cableway leading to the works lay-out, and the product sent to either a storage silo or direct to the concrete-mixing plant.

The material of minimum size (0·8 mm.) from the first washing screens was passed through two hoppers, cleaned in two washing machines, and conveyed, together with the product of the crushing mill, to the works. The cableway, about 2·5 kilometres long, divided into two parts by an intermediate station, carried the aggregate in containers, each having a capacity of 0·35 cubic metres; the discharging of the containers and the feeding of the concrete-mixing plant was controlled automatically.

The cement, in several containers, each having a capacity of 4000 kilograms, was carried by trucks to the construction plant and emptied into a hopper. From here it was passed through a closed conduit, then through a second horizontal conduit with a screw conveyor, and finally reached four cylindrical metal silos with a total capacity of 1,000 tons. Further screw-conveyors carried it to the silo feeding the weighing machines.

The proportion of the aggregates was regulated by five automatic weighing machines with the help of a conveyor belt. A sixth weighing machine regulated the cement; a volumetric batcher was used for the water.

The aggregates were fed into two concrete-mixers with a capacity of 0·75 cubic metre, through two hoppers, and into containers of the Blaw-Knox type, each having a capacity of 2 cubic metres. Flat trucks carried these containers either under the hooks of the derrick or to the cableway. The derrick, with a jib 50 metres long, was used for pouring the right abutment. The cableway, 290 metres long, with a moving tail-tower, was used for the construction of the middle part of the dam and the left abutment.

The output of the plant was about 260 cubic metres of concrete per 8-hour day.

As in the case of the other dams, special measuring instruments are installed (in addition to precision levelling and triangulation net-works), which will show the more outstanding movements of the rocks and the dam, namely, the following :—

A collimator, to check the movements of some fundamental points of the crest.

A coordimeter, to check the movements of the principal section of the dam.

Several strain gauges to check the most important and significant points of the dam.

Several deformometers and clinometers, to measure the movements and deformations in the proximity of joints, control drains, etc.

Several thermometers, to measure the changes of temperature of the concrete, the air, and the water in the proximity of the dam.

The upstream cofferdam and tunnels were completed in 1948, the pouring of concrete for the dam itself began in the spring of 1950, and the construction of the dam was to be completed in 1951. The contract was placed with an important Roman firm : the Società Italiana Condotte d'Acqua.

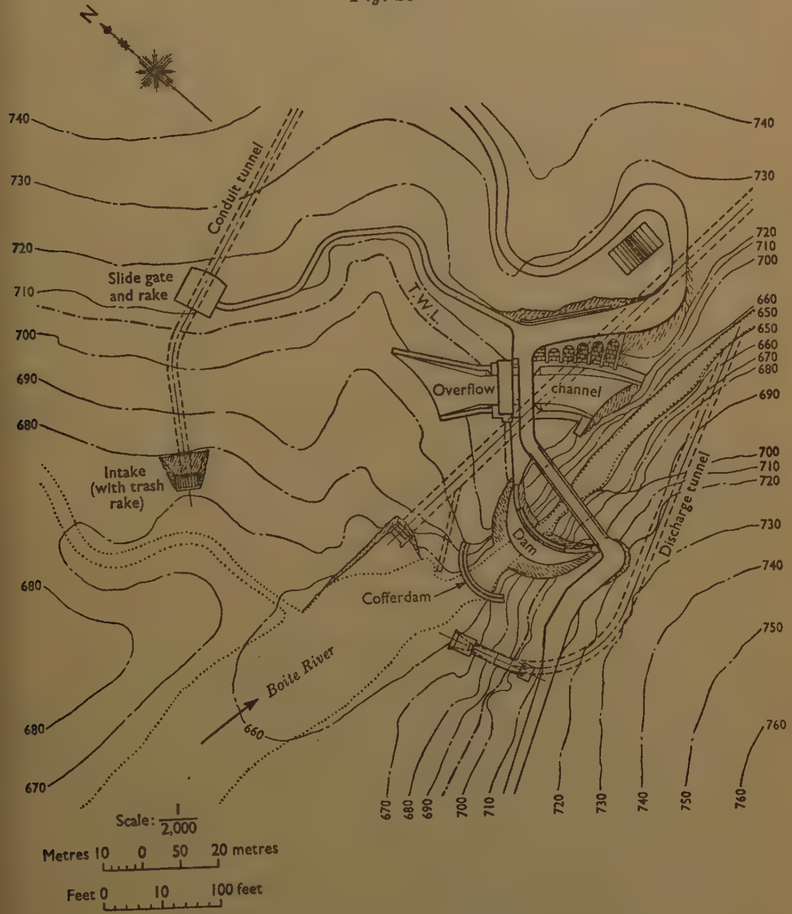
The Valle di Cadore Dam

See *Figs 16*, *17*, *18*, and *19* (facing p. 513)

The Valle di Cadore dam, consisting of an arch spanning a narrow gorge, naturally formed by erosion of the dolomitic limestone during the Ladinic period, is used to divert the River Boite.

The height of the dam is 58 metres with a total concrete volume of 4,000 cubic metres; the chord/height ratio is 0·425, and the thickness of the arch immediately above the base is 2·77 metres, not including the protection blocks mentioned below. The arch has two centres, one on the upstream face and one on the downstream face.

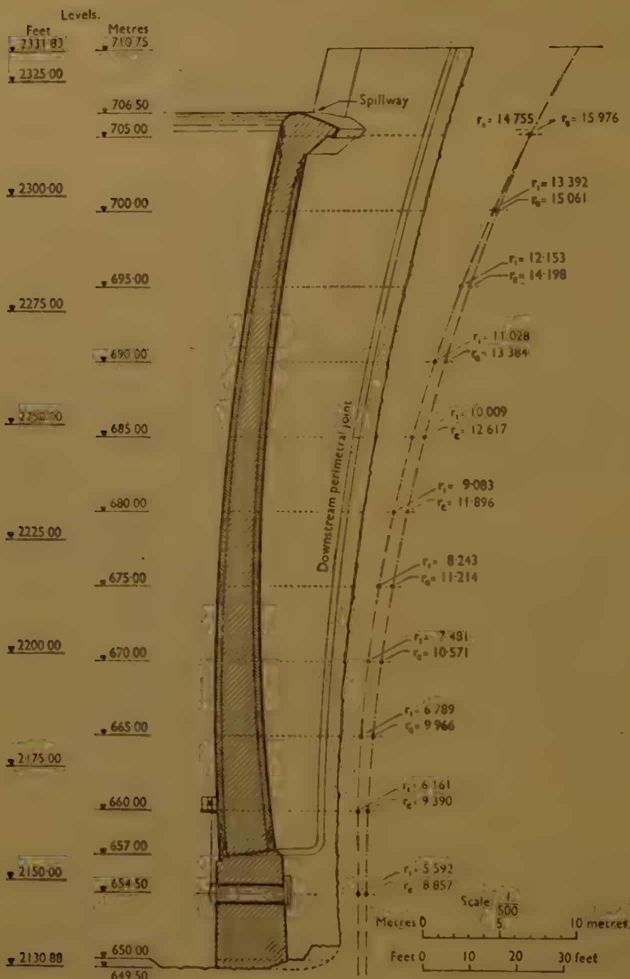
Fig. 16



THE VALLE DI CADORE DAM. GENERAL LAY-OUT

A notable feature is the protection of the structure against frost- and thaw-stresses, achieved by prefabricating the concrete blocks constituting the upstream and downstream faces. This should ensure the maintenance of the internal nucleus, the thickness of which is based on an analytical calculation. The construction was completed in 1950.

Fig. 17



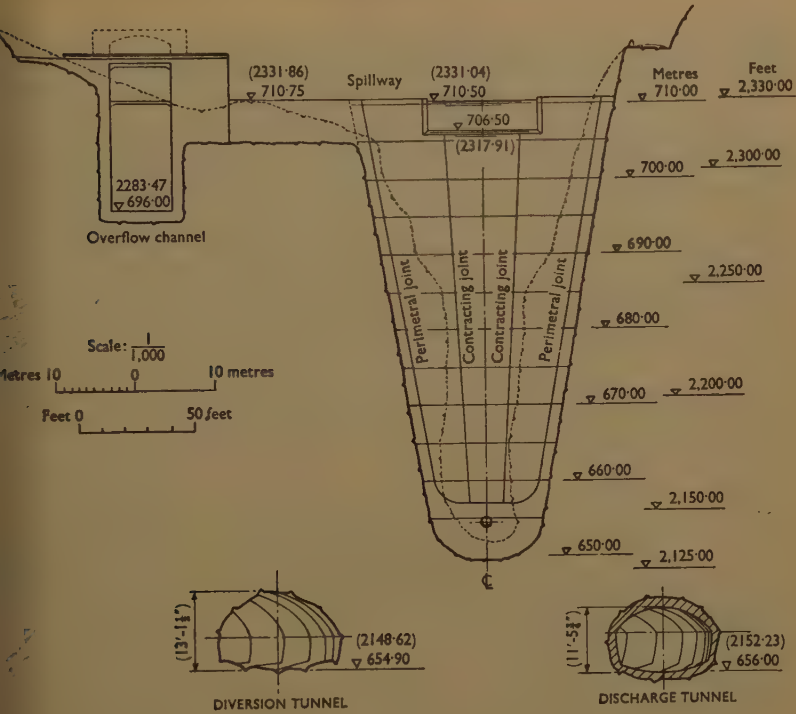
THE VALLE DI CADORE DAM. CROSS-SECTION

THE DAMS ON THE VAJONT AND THE MAÈ

It is the intention to start the preparatory works for the Vajont dam in 1952 or 1953. The tunnel for the water diversion has already been excavated.

The gorge eroded by the Vajont, a left tributary of the Piave, in the huge mass of Dolomitic limestone of the Jurassic period, which once divided

Figs 18



THE VALLE DI CADORE DAM.
SECTION ALONG THE MIDDLE FIBRE OF THE DAM

the basin of Erto from the valley of the Piave, is really striking. The rocky bottom of the gorge reached by drilling was found to be at an elevation 475 metres. With high water, the maximum level in the reservoir will reach 679 metres; therefore the height of the dam will be 204 metres. The chord/height ratio will be approximately ½.

As already mentioned, it is intended to include in the Piave-Boite-Vajont lay-out, a scheme to utilize the waters of the River Maè, one of the right tributaries of the River Piave. This could be achieved by the building of an arch-dam about 120 metres high across a narrow gorge of Dolomitic limestone of the Upper Trias, offering very good morphological and geological conditions.

CONCLUSION

In conclusion, it will be seen that when the whole of the Piave-Boite-Vajont project is completed, five dams of different proportions and construction, each representing a work of particular interest, will be grouped

together to form an integral system for the most economical exploitation of this catchment area.

Acknowledgement

The Author wishes to express appreciation for the enthusiastic co-operation and assistance from all concerned, including the departmental head, his staff, and the contractors and their workmen.

The Paper is accompanied by fifteen photographs and fifteen sheets of drawings from some of which the half-tone page plates, the folding Plates, and the Figures in the text have been prepared.

Fig. 20

THE VAL GALLINA DAM COMPLETED

Fig. 21

The Vajont Gorge

APPENDIX

Metric/English Equivalents (in order of occurrence)

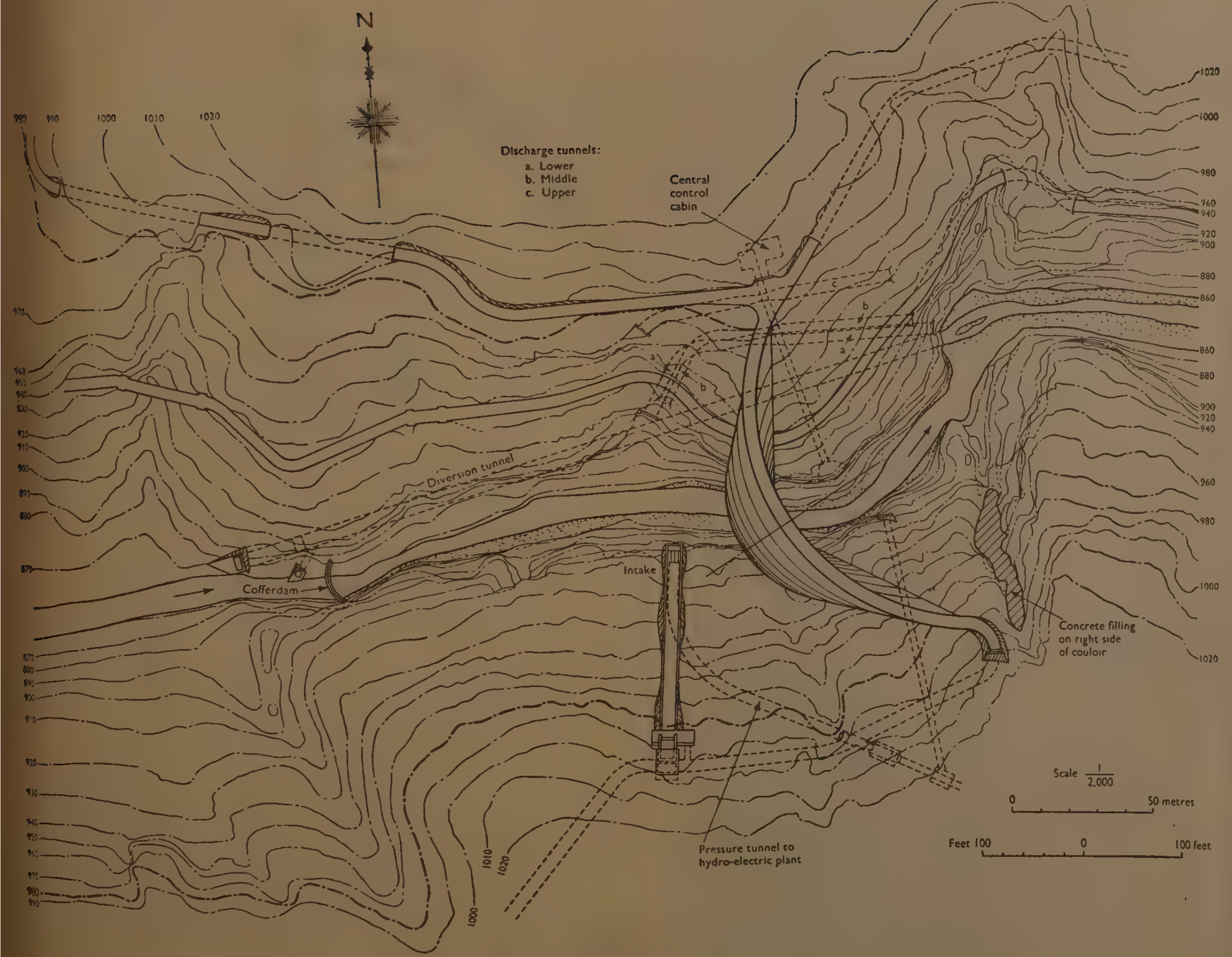
Page	Conversion
510	70,000,000 cu.m. = 91,500,000 cu. yds
	980·00 m. = 3,215·22 feet
	905·00 m. = 2,969·16 feet
	15 m. = 49·2 feet
	12 m. = 39·37 feet
511	134 m. = 439·63 feet
	130 m. = 426 feet 6 inches
	16 m. = 52 feet 6 inches
	3·15 m. = 10 feet 4½ inches
	20 m. = 65 feet 7 inches
	854·00 m. = 2,801·84 feet
	78 m. = 255 feet 11 inches
	910 m. = 2,985·57 feet
	50 m. = 164 feet
	15 m. = 49 feet 2 inches
	10 m. = 32 feet 10 inches
	900·00 m. = 2952·76 feet
	50 kg./sq. cm. = 711 lb./sq. in.
	8 kg./sq. cm. = 114 lb./sq. in.
	100,318 cu. m. = 131,206 cu. yds
512	63,000 cu. m. = 82,400 cu. yds
	3,903 cu. m. = 5,105 cu. yds
	350 kg./sq. cm. = 4,980 lb./sq. in.
513	528 kg./sq. cm. = 7,510 lb. /sq. in
	691 kg./sq. cm. = 9,828 lb./sq. in.
	18 km. = 11¼ miles
514	50 m. = 164 feet
	73 m. = 240 feet
	88 mm. = $3\frac{7}{16}$ inches
	40 mm. = $1\frac{9}{16}$ inch
	10 mm. = $\frac{3}{8}$ inch
	12 mm. = $\frac{15}{32}$ inch
	4 mm. = $\frac{5}{32}$ inch
	44 mm. = $1\frac{3}{4}$ inch
	1 cu. m. = 1·31 cu. yd
515	600 cu. m. = 785 cu. yds
	0·5 m. = 1 foot 7½ inches
	5 m. = 16 feet 5 inches
	2 m. = 6 feet 7 inches
	2½ m. = 8 feet 2 inches
	19,262 m. = 63,200 feet
	15,226 quintals = 1,500 tons
516	4·2 m. = 13 feet 9½ inches
	2·4 m. = 7 feet 10½ inches
	1·6 m. = 5 feet 3 inches
	1·20 m. = 3 feet 11 inches
	890·00 m. = 2,920·00 feet
517	121,000,000 cu. m. = 158,260,000 cu. yds
	13 m. = 42·65 feet
	70 m. = 229·66 feet
518	683·50 m. = 2,094·79 feet
	390·50 m. = 1,281·16 feet

APPENDIX—*Continued*

Page	Conversion
518	293 m. = 961 feet
	64,000,000 cu. m. = 84,000,000 cu. yds
	24 km. = 15 miles
	4·5 m. = 14 feet 9 inches
	18 km. = 11¼ miles
	58,000,000 cu. m. = 76,000,000 cu. yds
	6,000,000 cu. m. = 7,800,000 cu. yds
	4 km. = 2½ miles
	3 km. = 1⅞ miles
	2·5 km. = 1½ mile
	5 m. = 16 feet 5 inches
	830·00 m. = 2,723·12 feet
	13·00 m. = 42·65 feet
	250,000,000 cu. m. = 327,000,000 cu. yds
519	2·5 km. = 1½ mile
	27 km. = 16¾ miles
520	300 m. = 984 feet
	55 m. = 180 feet
	160·90 m. = 528 feet
	390 m. = 1,280 feet
	305 m. = 1,000 feet
	55 m. = 180 feet
	26 m. = 85 feet 4 inches
521	53 m. = 174 feet
	36 m. = 118 feet
	377,000 cu. m. = 493,000 cu. yds
	120,000 cu. m. = 157,000 cu. yds
522	0·01 mm. = 0·0004 inch
	100,000 kg./sq. cm. = $1{\cdot}422 \times 10^6$ lb./sq. in.
523	634·00 m. = 2,080·03 feet
	635·82 m. = 2,086·00 feet
	12 m. = 39 feet 4½ inches
524	200 kg/cu. m. = 337 lb./cu. yd
	1·5 m. = 4 feet 11 inches
	4 m. = 13 feet 1½ inch
	250 kg./cu. m. = 421 lb/cu. yd
	220 kg./cu. m. = 375 lb./cu. yd
525	135,000 cu. m. = 177,000 cu. yds
	120 cu. m. = 157 cu. yds
	27,000 cu. m. = 35,300 cu. yds
	40,000 cu. m. = 52,300 cu. yds
	600 m. = 650 yds
	150 mm. = 5·9 inches
	2,600 kg./cu. m. = 4,370 lb./cu. yd
	31 m. = 101 feet 8 inches
	2 m. = 6 feet 7 inches
526	1,100 cumecs = 38,900 cusecs
	3·5 m. = 11 feet 6 inches
	62,000 m. = 67,800 yards
	135 m. = 148 yards
	500 m. = 550 yards
528	5 m. = 16 feet 5 inches
	85 m. = 279 feet
	98,000 cu m. = 128,200 cu. yds
529	5,900,000 cu. m. = 7,720,000 cu. yds
	5·6 km. = 3½ miles

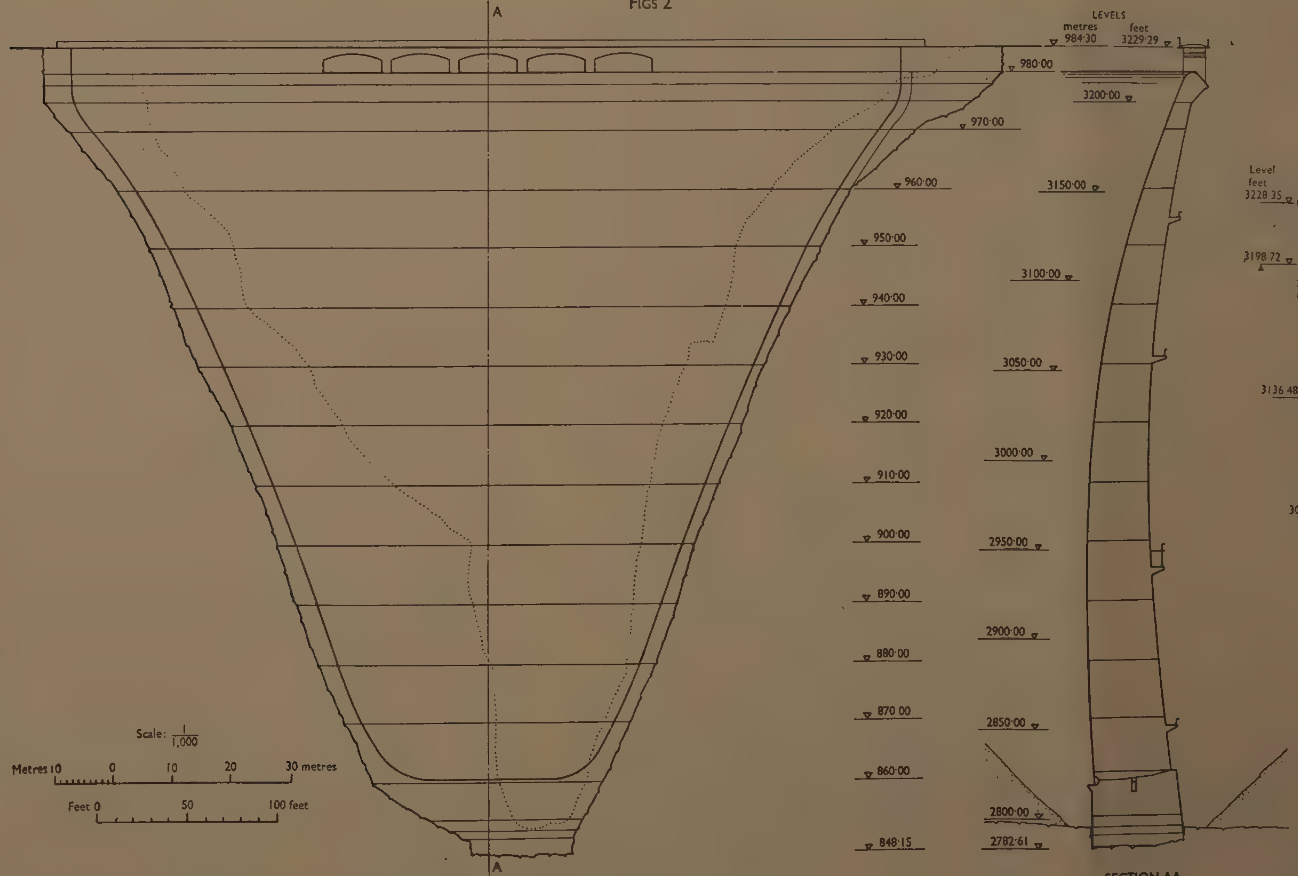
THE MOST RECENT DAMS BY THE "SOCIETÀ ADRIATICA DI ELETTRICITÀ (S.A.D.E.)" IN THE EASTERN ALPS

Fig. 1



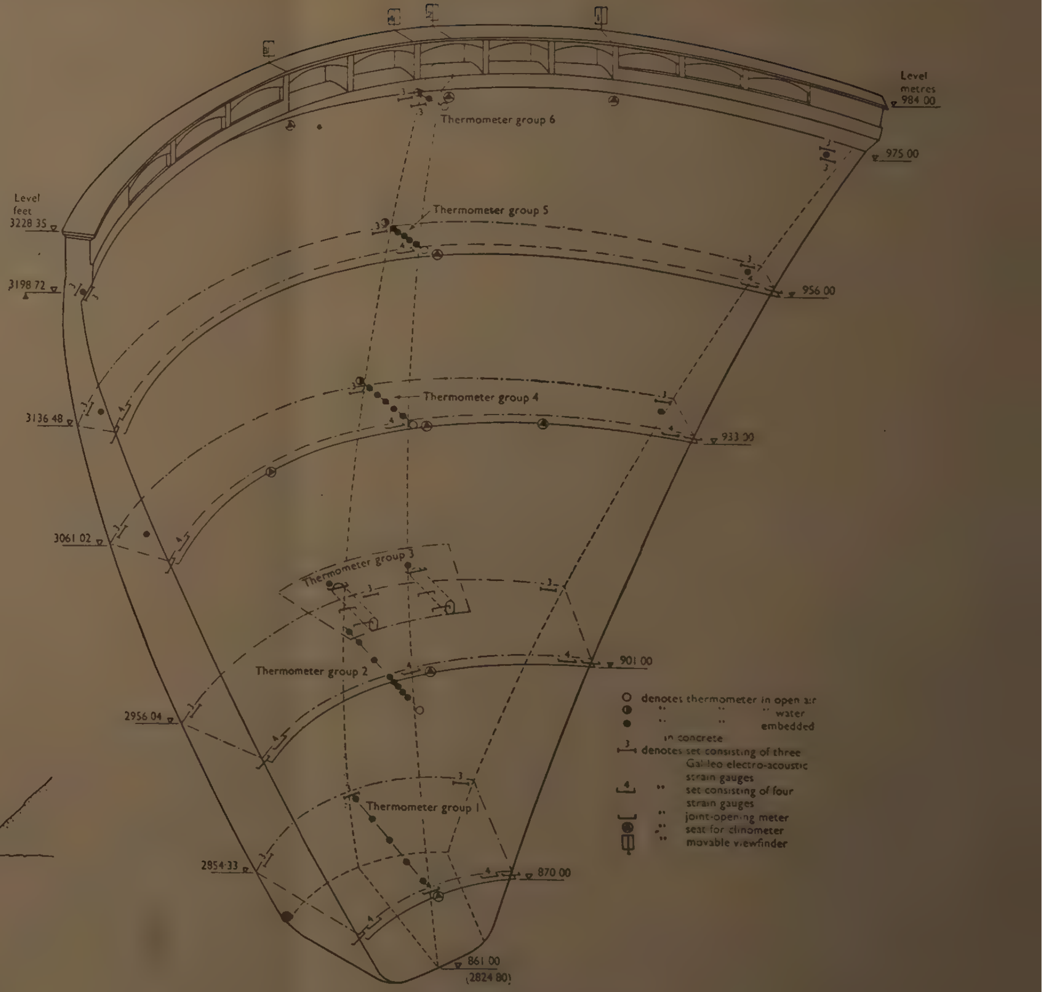
GENERAL LAY-OUT OF LUMIEI DAM

Figs 2



LUMIEI DAM

Fig 3



LUMIEI DAM — CONTROL INSTRUMENTS

CARLO SEMENZA

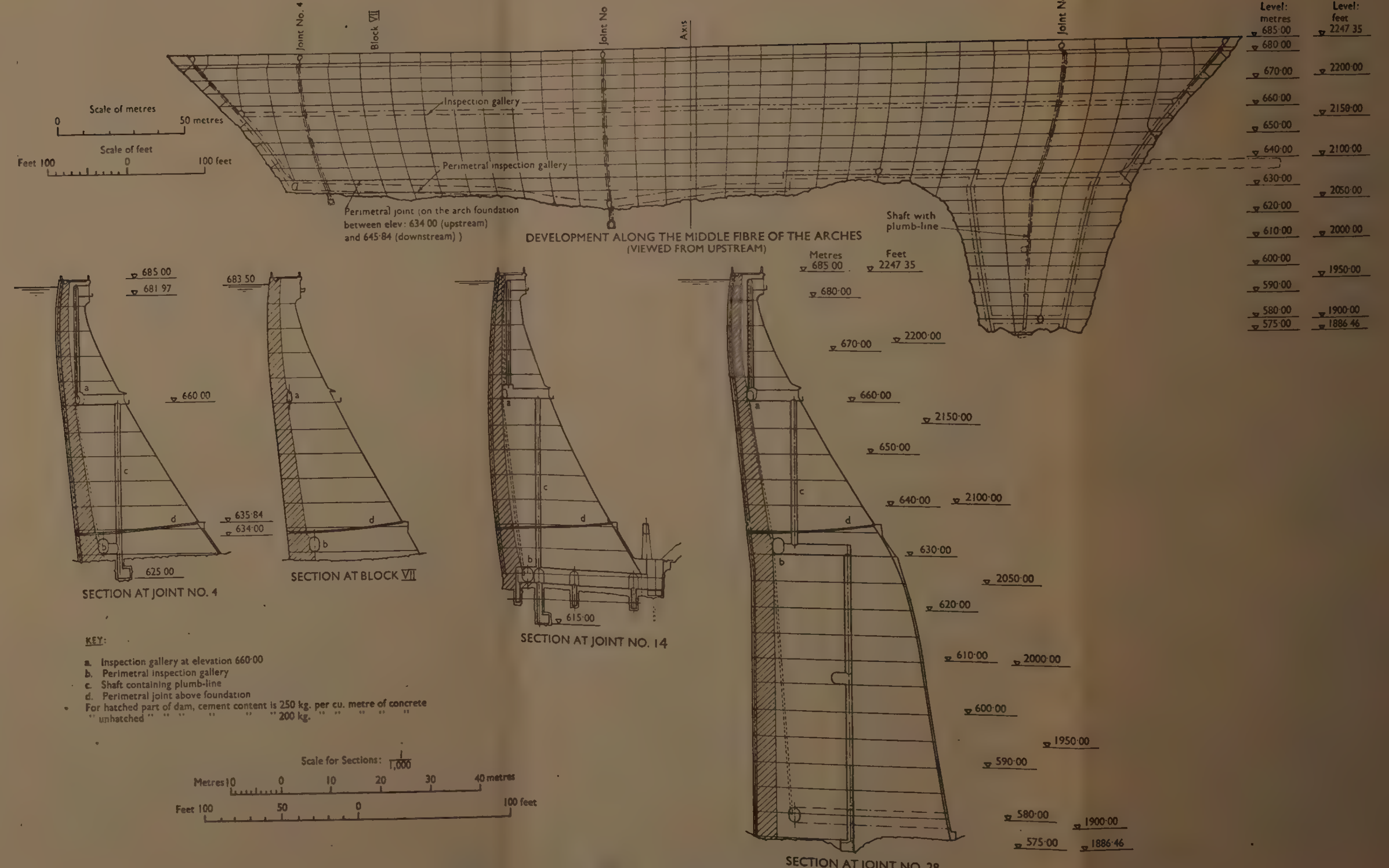
William Clowes & Sons, Limited: London

THE MOST RECENT DAMS BY THE "SOCIETÀ ADRIATICA DI ELETTRICITÀ (S.A.D.E.)" IN THE EASTERN ALPS

FIG. 7

PIEVE DI CADORE DAM — GENERAL LAY-OUT

FIGS 8



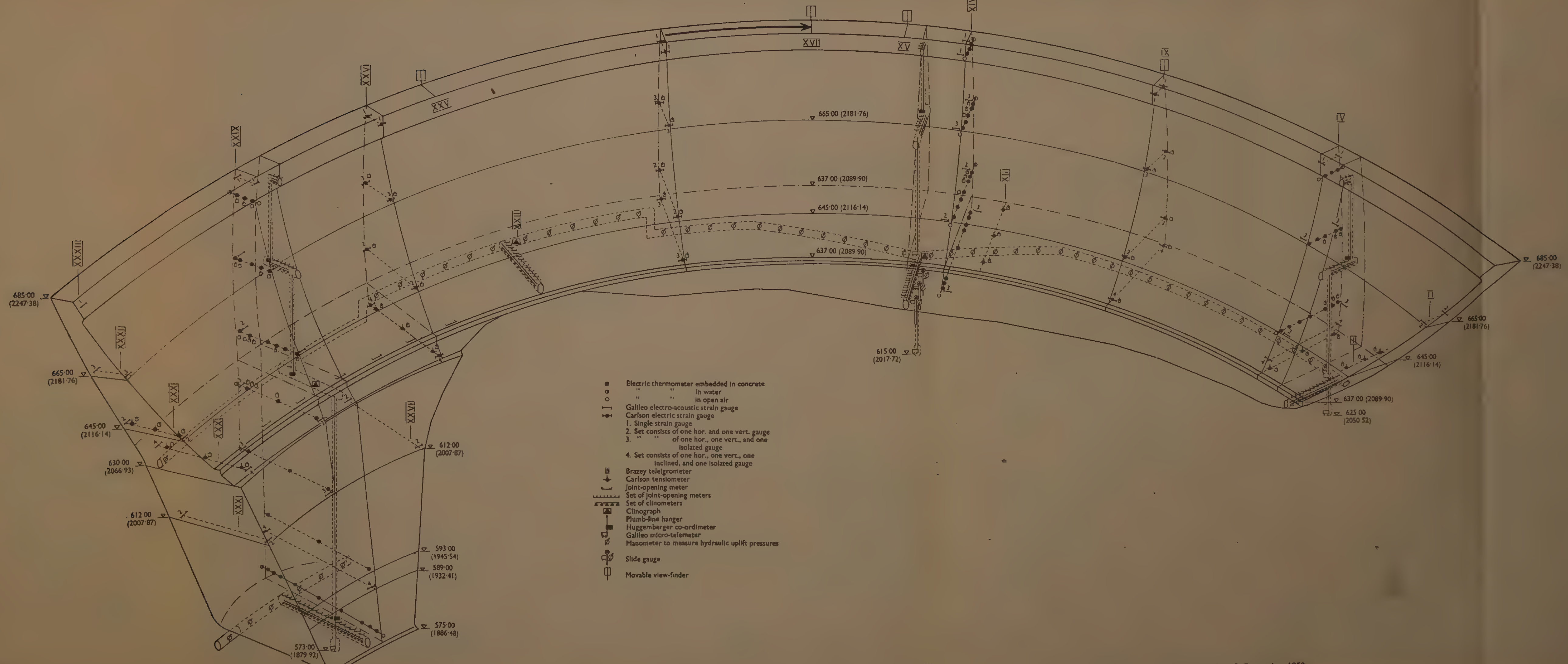
PIEVE DI CADORE DAM — DEVELOPMENT AND CROSS-SECTIONS

CARLO SEMENZA

WILLIAM CLOWES & SONS, LIMITED. LONDON

THE MOST RECENT DAMS BY THE "SOCIETÀ ADRIATICA DI ELETTRICITÀ (S.A.D.E.)" IN THE EASTERN ALPS

Fig. 9

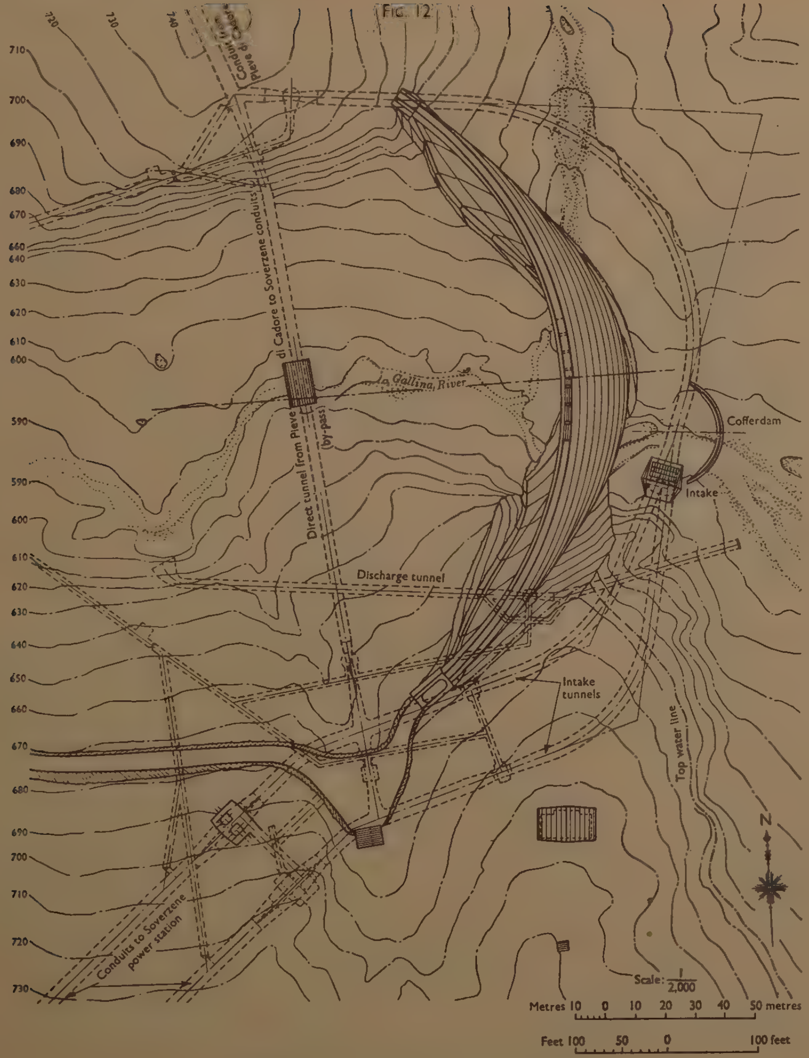


PIEVE DI CADORE DAM — CONTROL INSTRUMENTS

CARLO SEMENZA

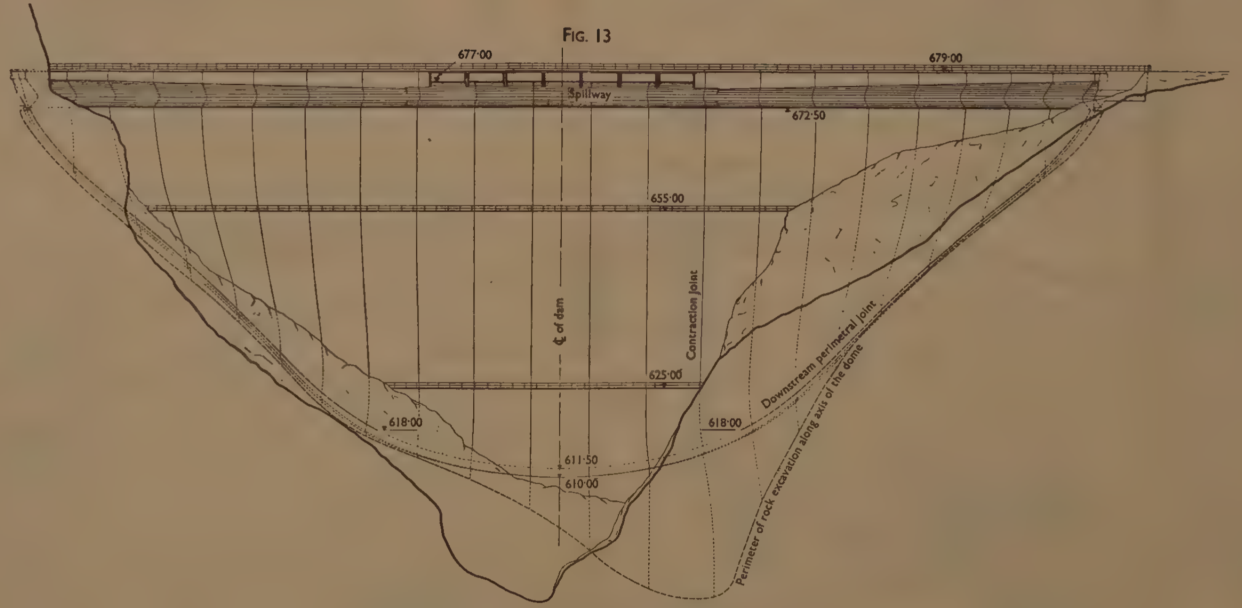
THE MOST RECENT DAMS BY THE "SOCIETÀ ADRIATICA DI ELETTRICITÀ (S.A.D.E.)" IN THE EASTERN ALPS

Fig. 12



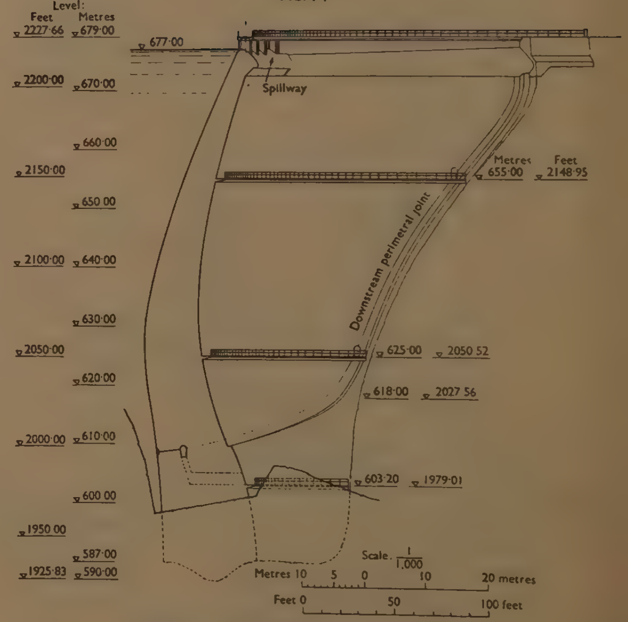
GENERAL LAY-OUT OF VAL GALLINA DAM

Fig. 13



DOWNSTREAM ELEVATION OF VAL GALLINA DAM

Fig. 14



CROSS-SECTION OF VAL GALLINA DAM

CARLO SEMENZA

William Clowes & Sons, Limited: London.

APPENDIX—*Continued*

Page	Conversion
529	250 kg./cu. m. = 420 lb./cu. yd
	32,201 m. = 35,214 yds
	3 cu. m. = 3·92 cu. yds
	21 m. = 69 feet
	40 mm. = $1\frac{9}{16}$ inch
	10 mm. = $\frac{3}{8}$ inch
	4 mm. = $\frac{5}{32}$ inch
	$2\frac{1}{2}$ km. = $1\frac{1}{2}$ mile
	0·35 cu. m. = 0·46 cu. yd
530	400 kg. = 882 lb.
	0·75 cu. m. = 0·98 cu. yd
	2 cu. m. = 2·6 cu. yds
	50 m. = 164 feet
	290 m. = 317 yards
	260 cu. m. = 340 cu. yds
531	58 m. = 190 feet
	4,000 cu. m. = 5,232 cu. yds
	2·77 m. = 9 feet
533	475·00 m. = 1,558·50 feet
	679·00 m. = 2,227·50 feet
	204 m. = 669 feet
	120 m. = 394 feet

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Mr Alan P. Lambert said that the analytical approach to structures—such as those described in the Paper—could be attempted only by an engineer of great skill and learning. Having read the Paper, he was sure that the Author had that skill and learning to a very high degree. His dams were daring structures—almost prototypes in each case—and the Author had shown that not only was he a great engineer but also that he had the courage of his convictions when he had been prepared to submit the results of his intellectual conclusions to the test of building those great structures, where any failure would be quite disastrous.

It was true that the Author's theoretical conclusions were supported by comprehensive and thorough tests— and it was wise and right that that precaution should be taken—but when the models themselves were examined one again found evidence of great skill. To be able to design a model of a dam and the apparatus needed to test it when the operative movements were of the order of 0·01 millimetre (something rather less than half a thousandth of an inch) showed a precision of quite exceptional quality.

Mr Lambert referred to one or two aspects of the design of the structures, and if the Author found his questions rather elementary he would forestall criticism by explaining that the designing of dams was not within his scope as an engineer. His first point concerned the perimetral joint, which seemed to be a most interesting device—a sort of concave saddle in which a saucer-like dome fitted. The Author had explained that it was a device to prevent any bending at the junction of the arch or dome with the abutment, but it seemed to Mr Lambert to cut across the conception of continuity which the Author had emphasized in a number of places in the Paper. The Author believed firmly in grouting—creating homogeneity—almost as though the dam were merged into the rock and one was not certain where it stopped being a dam and started being the rock; but across that continuity there was a curved cleavage, which presumably was necessary to prevent tension.

Mr Lambert did not understand why, in certain circumstances, the vertical cross-section overhung slightly upstream, and in other circumstances it overhung slightly downstream. Figs 8, Plate 2, showed a section at Block VII. When the dam was loaded, it obviously behaved as an arch, but when the dam was empty he would have imagined that that vertical section was subject only to the stresses of its own weight and, considering any horizontal section in that vertical section of Block VII, it would seem that the centre of gravity of the material above the section must be well outside the middle third of the section and might result in tension on the downstream face. In view of the fact that in the dam in question the eccentricity was upstream, it did not seem to him that that section would gain any support from the fact that the dam was arched in plan; the general slight movement, it seemed, would be away from the arch and not supported by the arch.

In Fig. 14, Plate 4, the inclination was shown to be quite pronounced downstream, and again when the dam was empty he would have thought that the centre of gravity of the material was outside the middle third, and in fact in some cases outside the dam altogether, which would perhaps introduce tension on the upstream face. In the particular dam in question, he could quite see that very little deflexion was necessaıy before the dome came into play and supported it; but if tension was present on the upstream face, and if the reinforcement referred to was there to deal with it, was there any risk of the concrete opening in the upstream face sufficiently to let the water reach the steel?

There were several references in the Paper to the provision of instruments to measure movements in the rock in the sides of the valley. Could the Author say whether those anticipated movements were a phenomenon which took place only when the dam was filled for the first time, or was it considered possible that they might extend over a prolonged period? Bearing in mind that the modulous of elasticity of the rock was much inferior to that of the concrete, was there any risk of the rock displaying

he phenomenon known as "creep" in concrete, amounting to a slow lastic yield? If that type of movement took place in the rock, Mr ambert would have thought that it might result in a re-adjustment of tresses in the dam itself which it would be impossible to calculate, and which might be very dangerous.

On the practical side of the Paper, he had been very interested to see that it had been found possible to make the sand fraction of the concrete rom the crushing of the rock. His own experience was that that was very ifficult, particularly when dealing with the igneous rocks which were perhaps more common in Great Britain than in Italy. Could the Author say if that sand fraction had been produced because of the nature of the limestone or of the type of crusher employed? In Scotland sand was now being carried a long way at great expense, because it had not been found possible, he thought, to make a satisfactory sand-fraction from the crushings of the rock. Was it the practice in Italy to wash rock aggregate after crushing to remove the "flour" usually present on the surface of the crushed particles?

Mr Lambert noted that in the four dams described in the Paper, the maximum size of aggregate ranged from 40 mm. to 150 mm. Would the Author say whether he was in favour of the use of large-size aggregates; and when using aggregate of 150 mm. size, did that introduce any practical difficulty into the process of mixing and placing the concrete?

The setting of the shuttering for those curved structures was surely a difficult problem, because it would not be possible to lay down the reference line on each lift as is usually done with a straight dam. What precautions had been taken to ensure that the profile of the dam, both in plan and vertical section, was worked to accurately?

The cofferdam shown in Fig. 7, Plate 2, was of great interest, but why was it necessary to build a cofferdam more than 100 feet high? Was the water impounded behind that dam to the full height, and if so, was that for the purpose of creating a head to pass the water through the diversion tunnel? British contractors might have felt inclined to construct a larger tunnel rather than risk the impounding of water at that height with so slender a structure as that arched cofferdam.

Finally, would the Author say whether the design of a temporary dam of that nature, 100 feet high, would be left to the contractor, or would it form a part of the considered design for the main scheme?

Mr G. M. Binnie referred to the geology of the sites. It seemed that the Author had been very fortunate in the geology of the limestone, considering what had happened in other parts of the world. One of Mr Binnie's colleagues had been concerned with the construction of an 80-metre-high arch dam in Andalusia, in Spain. There had been some doubt about the permeability of the reservoir area and so the dam had been built to only half height—40 metres; but then a tremendous flood had occurred and the dam had been topped, so it had been decided to complete

it to the originally intended height of 80 metres. The reservoir had never filled again to a height of more than about 10 metres, and the dam still stood as a monument of engineering but not of reservoir watertightness.

In another case, in Camarasa in Spain there had been a vast leakage problem of the order of 300 to 400 cusecs, and an extensive grouting programme had been necessary to reduce it. It had been brought down to about 90 cusecs after sinking boreholes of depths ranging up to 400 metres over a length of about a mile.

The Author had obviously been very much more fortunate, but he made one remark which was puzzling when, on p. 510 he had stated that the permeability of the limestones was usually across the valley. It would be interesting to know why that was so; it seemed to Mr Binnie very odd and he could not understand why it should be particularly across a valley instead of down the valley. It would seem that the dolomite was horizontally banded; was that so? No mention had been made of solution caverning; did that exist? For example, in the Tennessee Valley sites (in limestone) it had been found that solution caverning occurred down to depths of 250 feet below the water, but it would seem that the Author had not been troubled with it. Could the Author say how the grouting programme had been planned, what it was based on, and how the necessary amount of grout had been computed?

One of the most interesting features of the dams described was the importance attached to the symmetry of the structures and to avoiding the twisting moment which would otherwise occur. A considerable amount of excavation had in one case been undertaken—63 per cent of the total volume of the concrete.

The Lumiei dam had a remarkably thin cross-section in relation to its height. Perhaps the most outstanding case was the Pieve di Cadore dam, where the valley was so wide that one might expect the normal type of gravity dam to be adopted, but, in spite of its great width, the Author had been able to apply arch principles, with a substantial saving in cost. In order to do that, the perimetral joint which had been mentioned had been used, but it was not clear from the text how it was constructed, and any information which could be given about that would be of interest.

The perimetral joint, presumably, enabled the upper part of the structure to be treated quite separately from the lower part. By means of the perimetral joint, which was along the plateau and at half height across the gorge, it had been possible to get symmetry, or near symmetry. A fairly common type of valley formation was one in which there was a vertical cliff on one face and a gently sloping bank on the other. In such a case as that it would seem that it would not be possible to get symmetry except at exorbitant cost; did the Author agree with that view? It would seem to be necessary to adopt a different method in a case of that kind.

Another interesting point brought out in the Paper was the importance attached to continuity, that was to say, to the smooth and even distri-

bution of stresses within the structure itself and from the structure to the rock. A considerable amount of attention and thought appeared to have been given to the foundation conditions. On p. 522, the Author had referred to the modulus of elasticity of the rock being much inferior to that of the concrete, and to the percentage of the load absorbed by the arch as being greater than had been expected. The Author had referred to the fact that the calculations were based on a modulus of elasticity of the limestone of 100,000 kilograms per square centimetre, which was approximately equivalent to 1,400,000 lb. per square inch. At one limestone dam site in North Iraq, experiments had recently been carried out to determine the moduli of elasticity of the rocks by firing small charges in boreholes and measuring the wavelengths by geophysical means. A very close relation existed between the longitudinal wave-velocity and the Young's Modulus of the rock. That method of determination did not give the initial yield of a rock under static load nor the permanent yield with time, but it gave very quickly and easily the elasticity of the rock material. At that particular site the rock strata were nearly horizontal and consisted of very massive dolomite overlain by thinner banded limestone on the higher levels. In the dolomite, the moduli of elasticity were 7,500,000 lb. per square inch in a horizontal direction parallel to the jointing and 3,500,000 lb. per square inch in the vertical direction. In the limestone, the figure was 1,300,000 lb. per square inch in the horizontal direction, which was very close to the Author's figure, but less than 1,000,000 lb. per square inch in the vertical direction. When one compared those figures with the modulus of elasticity of the concrete in the dam, which was of the order of 3,000,000 to 3,500,000 lb. per square inch, it seemed that the rock might in some cases be more rigid, as in the case of the dolomite, where the figure was 7,500,000 lb. per square inch, and in other cases more elastic (as in the Author's case) than the concrete, and each case must be considered on its merits.

Mr Robert Carey said that he assumed that the object of the perimetral joint was to eliminate cantilever action in the dam, and hence to eliminate tension stresses. However, in the description of the Piave dam on p. 522 the Author had stated that the percentage of the load absorbed by the arches proved to be much greater than had been expected. It would be interesting to know how it had been expected that the load would be absorbed. Was it by cantilever action? In dealing with the Val Gallina dam, the Author had stated on p. 528 that "Owing to the shell-shape, a considerable part of the load is absorbed by the cantilevers." Would the Author clarify those points? In dealing with the Lumiei dam, it was stated that the design was a series of horizontal arches, 10 metres thick, and there was no mention there of any cantilever action. Mr Carey wondered whether that dam had been actually constructed as a series of rings, so that the arches would act independently of one another as true arches without cantilever action.

On p. 511 the Author had stated that the joint between the dome perimeter and the saddle in the case of the Lumiei dam was made watertight by means of several layers of asphaltic sheets and a staunching piece. In Great Britain much reliance was placed on copper sheeting for staunching-pieces, but copper was becoming very scarce and expensive. Had copper been used in the dams which the Author had described, or had something else been employed? The joints had then been grouted, and it would be useful to have some information on the method of grouting and on whether grouting pipes were built into the dam. How had the Author overcome the apparent difficulty of having layers of asphaltic sheeting in the joints, and how had that affected the grouting?

The grading curve derived from Table 2 seemed to flatten out at the 4–10-millimetre size, which seemed to indicate a discontinuous grading. Was that discontinuity in the grading deliberate? If so, it had apparently been very successful, because the compressive strengths of that concrete were remarkably high. Some very high strengths were given for blocks cut from the dam, but the age of those blocks was not stated, and it would be useful to have that information.

On the same point, Table 5 dealt with the Piave dam, and it would seem there that there was an excess of the middle sizes in the grading rather than a scarcity. No strengths were given, however, for the concrete in the Piave dam, and it would be interesting to know how the strength of that mix compared with the strength of the mix for the Lumiei dam.

The temperature rise was stated to be 31° C. in the Lumiei dam, and by the use of the "Plastiment" it had been reduced to 15° C. in the case of the Piave dam. The Author had stated that the normal temperature-rise in dams was 30° to 35° C. In the case of a mass-concrete gravity dam recently constructed in Scotland the temperature rise had been controlled to 20° C. without any addition to the concrete, merely by means of careful placing and controlling the temperature of the concrete when placed. In the Author's case, had the placing been slowed down in any way to control the temperature, or had the rate of placing been uncontrolled?

Mr P. O. Wolf observed that the "S.A.D.E." had built their dams in some magnificent mountain scenery, and he greatly admired the grace of the structures which was so strikingly shown in the Author's photographs.

The Author had mentioned the Italian Regulations of 1931, with which the construction of arch dams and domed dams apparently had to comply —even today. Did those Regulations really insist on a design analysis which considered the water load to be carried entirely by the horizontal arches? If so, the load taken by the Author's "inclined arches" of principal stress would be carried with a higher margin of safety. Or, in other words, the load carried by the so-called vertical cantilever elements (which in the Author's domed structures were really vertical arch elements)

would reduce the load to be carried by the horizontal arch elements. Although those structures looked extremely slender, could it not be argued that, owing to the observance of the Regulations, those dams had in fact a very generous factor of safety and could perhaps have been even more slender?

Considerable interest had already been expressed in the Author's figures of concrete quality, and Mr Wolf was looking forward to hearing the Author's explanation of the remarkable strengths listed in Table 3 achieved with a concrete containing about 4 cwt of cement per cubic yard, as shown in Table 2. The Author had used aggregates up to 3-inch size in his slender structures, and up to 5-inch in his thicker structures, and Mr Wolf asked for his views on the limiting size of aggregates to be used in future, bearing in mind questions of strength and temperature control as well as workability and economy.

It was interesting to note that, in Italy as in Great Britain and the United States, large concrete-mixing plants had been constructed for large dams. The size of plant giving optimum economy appeared to be one which could mix all the concrete required in approximately 2 years, that was, in two working seasons.

Reference had been made to the perimetral joint between the concrete saddle or plug, which was designed to be part of the rock foundation, and the slender dome. That joint was shown to be of uniform thickness from the upstream face and did not give the appearance of a rocker bearing. The dome was clearly so slender that, under load, its rim would be subject to appreciable rotation. Either the upper surface of the joint would have to move towards the water, which was unlikely, or the downstream edge of the joint would carry most of the thrust and give considerable eccentricity of loading.

Mr Binnie had made an interesting contribution to the question of Young's Modulus (E) of the bed-rock, and Mr Wolf wished to draw attention to a further aspect of that matter. He had seen the Author's original manuscript where the figures for E were quoted as 400,000 to 600,000 kilograms per square centimetre (say, 5·5 to 8·5 million lb. per square inch) for test cores taken from the rock, which was higher than for the concrete itself. Tests on the completed structure before grouting showed values of E quoted as 20,000 to 30,000 kilograms per square centimetre (say, 0·3 to 0·4 million lb. per square inch) and it would be interesting to know whether shrinkage and creep had been allowed for in computing those figures. Other figures for the modulus of the rock, after grouting around a tunnel, were quoted as 50,000 to 60,000 kilograms per square centimetre (say, 0·7 to 0·8 million lb. per square inch). That range of values which were a measure of movement under load of the foundation or springings would make it clear how difficult a stress analysis must have been.

On p. 520 the Author had briefly touched on his alternative designs of

the Pieve di Cadore Dam. From Fig. 7, Plate 2, it would appear that the deep gorge could have been closed by a thin arch dam supported on its left by a buttress taking the form of a straight dam connecting the arch to the left bank of the valley. There were precedents for that idea of combining arch and gravity designs. There were also some dams in existence which consisted essentially of two structurally independent "wings," either arched or straight; if the Author found it necessary to construct a separate buttress to take the arch thrust, he could have built the left-hand "wing" in the form of a straight buttress-dam or of a straight hollow gravity-dam, the cost of which should not have been more than that of the longer gravity-arch. By the adoption of suitable upstream and downstream batters the excessive tensile stresses mentioned should have been avoided without much extra cost. That considerable tensile stresses were permissible was clear from a study of the cross-section in Figs 8, Plate 2.

The results of measurements on the complete Pieve di Cadore Dam were of great interest, and Mr Wolf therefore asked the Author for a brief elucidation of the diagrams contained in *Figs 10*. He asked which joints opened at the downstream face under steady maximum water-load, as was shown in the left-hand graph; what were the air and water temperatures at the time when the concrete temperatures in the dam were registered; and what were the datum and the direction of each vertical displacement plotted on the third graph?

The Paper, though complete in itself and of outstanding interest, would increase in value if the Author could add a bibliography covering the publications on which he and his colleagues had based their designs and those in which they described their own analysis and model testing of the dams and the design of associated works.

Dr Charles Jaeger observed that the valuable and thorough Papers published by the Author in Italian were highly appreciated in Great Britain. It was very desirable that engineers in all countries should follow the Italian example of publishing their results openly, making them available to their colleagues in other countries.

One of the slides used by the Author when introducing his Paper had shown the lines of principal stresses in the Val Gallina dam inclined against the banks of the river. The same configuration was known to exist in other dams. However, in a Paper published by Professor Oberti[1] showing fissures in the model during the tests made for the Lumiei dam, all the fissures were horizontal. Could any explanation be given for that discrepancy between the measurements made when the dam was not overloaded and what was observed when the model was tested to final breakdown?

[1] Professor Guido Oberti, "*Diga del Lumiei, Criteri di Progetto e Studi Sperimentali*" ("Design Data for the Lumiei Dam, and Experimental Investigations"). *L'Energia Elettrica*, vol. 25 (1948), No. 9, Fig. 16. Also in *L'Impianto Idroelettrico del Lumiei*" ("The Lumiei Hydro-Electric Plant"). Società Adriatica di Elettricità, Venice, 1950.

In conjunction with that question of the shape of the principal stresses, was the Author familiar with the method of calculation adopted by Coyne, who had stated that he was calculating the arch dam at Tignes (France) with a new "inclined arch method," somewhat different from the method which had been used by the Società Edison for one of their dams? Coyne had, however, never published anything apart from a few lines in the book by A. Bourgin.[1] It would be interesting to know whether more was known in Italy than in Great Britain about the method in question.

The Italian dams had, in general, been tested with models made of plaster of Paris, though Dr Jaeger believed that one dam had been tested on a model in concrete. He knew that in other countries cork and rubber had been used for similar tests. Had the Italian engineers any experience of the results which were obtained with material other than plaster of Paris?

With regard to the aggregate, two dams had been built with a so-called discontinuous granulometry, and he wondered whether in Italy that had been used, and whether the Italians had some experience of air-entrainment products added to the concrete, different from the Plastiment, which was not an air-entrainment product.

Professor Guido Oberti observed that all questions relating to joints—not only the perimetral joint, but also radial joints—had been studied by means of model-tests to prevent the effects of cracking, because it had been desirable to do whatever was possible to prevent cracking in all those great concrete structures—especially on the upstream faces. Cracking might depend not only on the effect of water load and the dead load, but especially on the effects of shrinkage and of temperature, which were very difficult to allow for in any calculation. It had been considered better to try to prevent such effects, and to deal with the possibility of movement by means of joints. The use of joints in the dams had therefore been the subject of intensive study.

The perimetral joint divided the structure into two parts, the dam proper and the saddle which was fixed or clamped to the rock. That was a good practice and had made it possible to use ordinary calculations for the symmetrical part of the dam. The plugs had been designed with the same aim, the first idea being to regularize the form of the valley by means of the saddles and plugs. Those saddles and plugs were very difficult to calculate, and that was why great use had been made of models. The perimetral joint modified the stresses in the cantilevers and in the arches. That was rather a difficult question, especially when it was necessary to study domed structures and there, too, models were of great assistance.

Dr Jaeger had asked about the material used for the models. For several years plaster of Paris had been employed, but it was rather difficult to use, especially when the models were large and thick, because the plaster

[1] A. Bourgin, "*Cours de Calcul de Barrages*" ("Calculations for Dams"). Eyrolles, Paris, 1948.

took a long time to dry and the interior was not uniform. That was the principal reason which had led to the use of a special mixture for the most recent models. It was a special concrete in which the aggregates were volcanic stone from the island of Lipari; such stone was very cheap in Italy and was therefore very useful for laboratory tests. Using volcanic stone, several tests had been made with different percentages of cement, and it was now possible to construct a model of that special mixture having a large variation in the ratio of its modulus of elasticity to the modulus of elasticity of concrete. In some cases the ratio is as low as one-tenth. Furthermore, in the large models for the Piave dam it had been possible to include the mountain rocks in the model, incorporating the same ratio as existed in nature, obtained by means of research directly made in the field by means of special devices, particularly for the limestone. The necessary information had been gained by measuring the deflexions (in all directions) caused by pressure in a large tunnel which had been excavated in the interior of the mountain.

The movement of joints had been controlled by means of special instruments, particularly on the Piave dam. Admittedly these tests had taken considerable time, but it had been established that along the surface downstream and to a depth of 2 or 3 metres, there was continuous movement of the joints near the surface. That also had to be incorporated in the calculation when possible.

The line of the direction of stresses determined by means of model-experiments as shown on the Val Gallina dam, was downstream, in the form of ideal arches which were inclined towards the abutments; upstream, however, the line was almost horizontal. The latter had not been completely measured on the Val Gallina dam, but it was known from experience elsewhere. Some calculations had also been made on the Val Gallina dam, by means of the theory of inclined arches (the "arcs plongeants" of French engineers). That was a method employed especially by French engineers. Those results had been compared with the results obtained from models and from other calculations.

Professor Dino Tonini observed that the controlling and gauging of a dam required a very long study to obtain results. There were several influences acting on the dam, including climatological conditions, and it was not easy to separate them. The question had been asked as to whether it was too early yet to arrive at a general conclusion. He thought that it was now possible to say that, first, there was very good agreement between theoretical and experimental results for determining the thermal behaviour of the dam. Secondly, there was good agreement between theoretical and experimental results for the movement of the dam caused by external loads acting for a short time (during rapid filling or emptying of the reservoir with a steady temperature, or during change of temperature with a steady load). Thirdly, the abutments and the foundation deformations had a great influence on the movement of the dam. Fourthly,

whilst temperature was a leading factor in the movement of the dam, the same could not be said of the stresses.

*** **Professor A. L. L. Baker** inquired whether the use of an elastic net had been tried in order to determine the best form of dam and whether the Author considered it might be a useful method. Professor Baker suggested that a model of the valley section could be turned over to a horizontal position and loads suspended from a net across the valley with guides to cause the loads to act radially, the loads being of suitable value to represent water pressure. The form of the net producing minimum tension might then be found by trial—the same shape, but convex upstream, producing no bending for water pressure only. By shortening the strings of the net alternately and in other ways, the effects of shrinkage, creep, and temperature variations might be studied. Having determined the best shape of the dam in that way, a model might then be made and tested as described in the Paper.

The Author, in reply, wished to make it clear that much of the work described in the Paper had been done by his engineers and his answers could be authoritative only after consultation with them.

Mr Lambert and others had apparently detected some incompatibility between the idea of continuity and the adoption of joints—the perimetral and the intermediate ones. The Author felt that he had not succeeded in making his thoughts clear enough ; by continuity he meant a gradual variation of the geometrical parameters which defined the characteristics of the dam.

As he had already pointed out in the Paper and elsewhere,[1] he meant that the structure should be able to absorb, transmit, and distribute, evenly and without sharp variations, the stresses in all its parts ; and that the same gradual transmission should be ensured from the structure to the rock foundation. He certainly did not mean absolute continuity or monolithicity of the structure (though ideally desirable), because (as already pointed out in the Paper) joints were an unavoidable necessity, owing to the features of the materials that had to be used and the inherent constructional exigencies. However, he thought that every constructional element should be studied with regard to continuity ; also the joints, for instance, at which sharp variations of planes and directions should be avoided.

Even on the assumption that the structure could not be continuous in an absolute and mathematical sense, it should consist of a system of solids, each of them conceived as a continuous element.

The overhangs and eccentricities referred to by Mr Lambert had been thoroughly studied with the exact aim of balancing the stresses ; an

*** This contribution was submitted in writing.—SEC. I.C.E.

[1] "*Les barrages de la Società Adriatica di Elettricità en Vénétie*" ("The dams of the S.A.D.E. in Venetia"). Bull. Tech. Suisse Rom., Lausanne, 1949.

attempt had been made to reduce to the smallest possible areas and to the lowest values the tension stresses on the upstream face, admitting of course higher values on the downstream face of the dam. Mr Lambert's remarks about the cross-sections of the Piave and Val Gallina dams were correct in a theoretical sense; however, it had to be remembered that only an actual crushing of the compressed areas might determine the setting up of considerable tension stresses on the opposite face; such a condition was in fact not even approached and so was really no risk at all. The Author reminded Mr Lambert that both the upstream and downstream faces of the dams (except the Piave dam) were reinforced.

Mr Lambert had expressed his concern at the possibility of slow plastic yield of the rock. The Author did not believe that a "creep" similar to that of concrete occurred in the limestones with which he had to deal. In such rocks, there had been noticed an elastic settlement consequent upon the first filling-up of the reservoir; after that, very much reduced movements had taken place corresponding to the subsequent emptyings and fillings, but deflexions had soon become negligible.

All those phenomena needed further controls and a thorough study, which would probably be possible in a few years' time.

The making of sand from the crushing of the rock was common practice in Italy, by means of hammer-mills or ball-crushers: it was, of course, easier with limestone and dolomitic rocks, but the method had been applied also with igneous ones (for instance at the Travignolo dam). The removal of flour was not a common practice, though it was desirable. At Piave, however, the amount of "flour" was excessive and it was removed by washing. The Author considered that removal of "flour" was absolutely necessary when more than 2 per cent of clay was present.

Mr Lambert's question regarding size of aggregates was referred to later in reply to Mr Wolf.

Setting-out and shuttering for double-curved dams were not easy and required particularly skilled men. However, Italian contractors had nowadays succeeded in training very skilful teams of surveyors and carpenters. It had been noticed that the cost of shuttering depended more upon the number of the steel panels required than upon the difficulty of placing them.

The height adopted for the Piave cofferdam (Fig. 7, Plate 2) had been necessary to ensure sufficient hydraulic head at the tunnel inlets for the required flow in the tunnels. A tunnel diameter large enough to give the same flow with a lower inlet charge would have been much more expensive. It should also be remembered that one of the tunnels (the upstream one), owing to the topography of the location, was situated at a much higher elevation; that, in itself, gave rise to an unusually high cofferdam. On the other hand, there was no risk in using such a slender structure even from the dynamic stresses set up by overflow water, because

it was very carefully studied and built (with proper reinforcing) and it abutted on solid rock.

The design of the cofferdam had been made by the Author's staff as a part of the considered design for the main scheme. Owing to its dimensions, it had been subjected, exactly like the main dams, to the control of the "Servizio Dighe" of the Ministry of Public Works. Usually, Author's company did not expect designs and analysis to be made by the contractors, except for the construction plants.

When designing and building the dams described in the Paper, the Author had been aware of, and had borne in mind, the Spanish examples mentioned by Mr Binnie. A similar case had also occurred in Italy many years before; it had therefore been possible to eliminate losses by means of ordinary grouting works.

Italy was fortunate in possessing some geologists who were particularly expert in problems concerning dams. The Author was especially grateful to Professor Giorgio Dal Piaz—former professor of Geology at the University of Padua—who had given excellent assistance for the past 40 years.

In general, it could be stated that any location for dams was a problem in itself from the geological point of view as well as from the morphological one, and it should therefore be studied from those particular viewpoints. There were, of course, limestones of very different ages and characteristics.

The Author's statement about transverse permeability was not originally his own; he had heard it about 15 years previously from a distinguished Jugoslav geologist, who had explained on that basis the design of a dam in an entirely Carsic district. That statement had been an interesting one, bearing in mind the particular conditions in which reliance had been placed upon it. The Author thought it probable that the water circulation in the rocky sides of a gorge tried to follow the shortest way to the bottom of the gorge itself—following the shorter cross-planes and not the longer ones, parallel to the valley. Water circulation did not necessarily follow the surfaces of layers (which were often closed and often approximately horizontal) but the small cleavages. At any rate, the Author had purposely excluded the case of Carsic ground (not assuming as absolute the opinion of the Jugoslav geologist) because in that case the water movement probably followed different ways.

On the higher parts of the sides of rocky valleys cut in the limestone, numerous parallel furrows could frequently be noted sloping downwards, that was to say, towards the bottom of the valley. Those were the so-called "*karren*" ("*campi carreggiati*"), and many examples of them existed in all limestone regions.

As the furrows deepened, the surface waters (both rain and melting snow) penetrated into the internal cracks of the rock and especially into those having a more immediate outlet. The outlet of course could only follow a downward direction, that was, towards the valley below, though

the path followed by the water was underground. In that way, the cracks gradually became active underground conduits, taking in water from the upper veins, and they sometimes developed into actual underground channels, which fed the well-known Carsic springs rising from the foot of the valley sides.

The underground water-system, in such cases, consisted of a series of conduits or channels, small and intermittent at first, but increasing gradually by their tributaries. The whole system, especially in the sections of greater flow (that was to say, in the lower ones) ran more or less at right angles to the axis of the valley, which represented the collector of the flows coming from the sides. No longitudinal underground channels, parallel to the direction of the valleys and connecting the veins coming down from the upper regions of the valleys, could exist in those cases where the Carsic phenomena were rather recent, and consequently not greatly developed; the various conduits reached the basic hydrological axis without any practical possibility of deviating their flow from one into another.

The situation was very different with a limestone rock in which the Carsic phenomena were deeply developed; in those cases the rocks were sometimes transformed into a sort of sponge. The internal waters followed many different directions (sometimes parallel to the main hydrological axis) and those conditions might cause dangerous leakages.

The same thing also occurred in the Carsic limestone highlands, in which deep and long valleys did not exist, and where there was no natural or other drainage of the Carsic rocks. Also in that case the underground waters followed various and irregular directions and fed the numerous springs that issued, with no apparent order, from the sides of the rocky masses, as if they were a layer of water and not a water system converging in single channels roughly parallel to one another and at right angles to the axis of the valley.

Without wishing to give to those suggestions a special value, the Author could only state that what he had seen in the Venetian glens apparently corresponded exactly, in several cases, to the above-mentioned diagnosis. In very rainy weather there was an abundance of springs issuing from the rocky walls at different elevations above the bottom; and that happened in valleys where, afterwards, reservoirs with very high dams had been successfully established. An example was the case of the Isonzo dam; initial investigation of the condition of the site on rainy days had been very discouraging.

The Isonzo dam was a big dam built by the Author in a district then in Italy but now in Jugoslavia. Perfect watertightness had been secured by means of a not very extensive grouting screen.

It was fortunate that solution caverning had never been found in the limestone of the Eastern Alps; if it had existed the problem would have been very serious.

For the Piave dam, about 10,000 tons of cement had been injected so far. A supplementary programme of groutings had been drawn up, with the object of reducing some residual losses on both sides, which although of no great importance, were undesirable.

In the locations of the Italian dams, the strata were mostly horizontal or slightly inclined: only occasionally did they have a steep inclination.

The grouting programme and the necessary amount of cement were usually based on a series of drilling and permeability tests pushed down into the rock to a depth corresponding to the height of the dam, and in any case far enough down to reach a value of the natural permeability inferior to the pre-established one. In several cases, preliminary tests of cementation were made—for instance, for the Val Gallina dam, as mentioned in the Paper.

The grouting programme, and especially the inclination and distance of holes, varied of course with the inclination of the layers. Due consideration should be given to the same inclination in the design of the dam, because the reaction of the rock varied according to it.

With regard to the symmetrical shape adopted for the Lumiei dam, the particular shape of the gorge would, in any case, have necessitated very deep excavations in order to build an arch dam with suitable radii and central angles, had it been symmetrical or not.

The right side of the torrent at that location tended in its lower part to diverge considerably from the axis of symmetry of the dam (see Fig. 1, Plate 1). Therefore, in order to build radial abutments, the excavations towards the upstream face would in any case have been extensive.

In the cases of complete asymmetry brought forward by Mr Binnie, it would of course have been difficult to achieve perfect symmetry; but, by using a proper plug and the perimetral joint (as for the Val Gallina dam, and with two other Italian dams—Ponte Racli on the Meduna of the S.A.I.C.I. and Val Travignolo of the SMIRREL), the conditions could be improved. In any case, symmetry was not an absolute statement and the Author attached greater importance to continuity than to symmetry.

However, the Author acknowledged that the many models tested had clearly shown that symmetry gave a more harmonic behaviour.

Generally speaking, symmetry was more convenient where the average values of the stresses were already high; in such cases, the addition of the twisting stresses due to asymmetry would cause the total stresses to become incompatible with safety limits.

From that standpoint, it might be possible in some cases to achieve symmetry and at the same time to obtain a substantial economy, notwithstanding the increase of the developed surface.

In the case of asymmetry, the dam was analysed by means of the Trial Load Method; model testing in that case was obviously very valuable.

The geophysical method of obtaining the moduli of elasticity of the rocks had also been used in Italy, where, as in Iraq, higher values had been

obtained, from which the instantaneous modulus (called, in Italy, the "dynamic modulus") resulted. From that modulus, therefore, there could be obtained a first parameter to be used as a comparative value. However, for a study of the static behaviour of a structure, the moduli determined according to static methods more nearly represented the real behaviour. In any case it was perfectly correct to assume, as Mr Binnie had stated, that "each case must be considered on its merits."

Turning to Mr Carey's remarks concerning the Piave dam, the Author pointed out that when discussing the expectation of a rather small percentage of the load to be absorbed by the arches, he had been referring to the analytical calculations of the design. In fact, the analysis had been carried out without considering the perimetral joint; that had resulted in more importance being attached to the share of the acting forces absorbed by the cantilevers. Another element increasing the importance of those forces was the hypothesis of a smaller yielding of the rock than the real one.

Therefore, the analyses had obviously been approximate. The model, on the contrary, had proved the contribution of the arches to be higher, because the influence of the perimetral joint had been allowed for and at the same time more probable hypotheses regarding the yielding of the rock had been formulated. It should be remembered that the rock yielding greatly affected that type of dam.

In the Val Gallina dam, the influence of the cantilevers, intuitively foreseen on the grounds of previous experiences and of the shape of the dam, had been confirmed by the model tests. The actual figures of percentage of loads absorbed by arches and cantilevers in both dams would be published later (probably in "*L'Energia Elettrica*"), together with other important data.

The Lumiei dam had been conceived and built as a curved plate and not as a series of rings. The analysis, on the contrary, had been made as if for independent rings, according, as already pointed out in the Paper, to the common elastic theory. The behaviour of the cantilevers, in that particular case, was of secondary importance, and the analysis had therefore been limited to a verification through the model.

The joints, *Fig. 22*, had been equipped with joint-covers and copper panels, which the Author considered essential; they had been grouted by means of holes drilled afterwards, and that system had also been followed for the other dams (except for the Piave, where steel grouting pipes had been placed in the dam). Both systems had been successful; the Author supposed that in a massive structure, like the Piave dam, the second system was more suitable. Layers of asphaltic sheeting had not been placed in the joints—only under the staunching pieces.

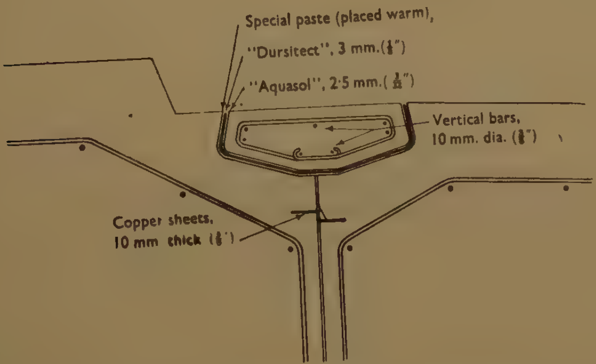
The grading curve (Table 2) had been adopted after many mixing tests; it had actually proved to be the best obtainable with the available aggregates. The slight discontinuity, practically compensated for by the

undersize elements coming from the 10–40 mm. silo, was deliberate. The blocks cut out of the dam were aged 6 months.

For the Piave dam the percentage of sand was raised, because the Dolomitic limestone produced a rougher sand than the one resulting from the Lumiei limestone; moreover, the concrete for the upstream face had to be perfectly impervious.

As Mr Carey had remarked, there was indeed an excess of middle sizes. But, as in the case of the Lumiei dam, excellent strengths had been obtained, probably because of: (*a*) the characteristics of the limestone aggregates themselves; (*b*) the reduction of the water-cement ratio

Fig. 22

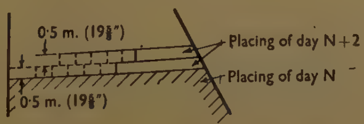


obtained through the use of Plastiment; and (*c*) the efficiency of vibration through the Notz apparatuses (much more efficient than the ones used for the Lumiei dam). In addition, the low-heat cement offered, in the long run, higher resistances than the types giving high strengths at the beginning.

As mentioned above, a final report on the complete results obtained at the Piave dam was being prepared; it would contain full information and would be available to anyone on request.

The reduction of the temperature rise had been mainly attributable to careful placing and to the lower percentage of tri-calcium silicate; that

Fig. 23



had been as much as 45 per cent during the hot months. Every second day, block layers, 1 metre thick, composed of two layers, each 0·5 thick, had been poured (*Fig. 23*). On hot days, the two layers had

been placed at the same time as shown in the Figure, in order to reduce the action of heat on the surface exposed to the sun. The corresponding layers on the adjacent blocks had been poured, after at least 3 days. The poured layers had been watered, the water having a temperature of 10 to 12° C. With regard to temperature, the Plastiment had only a secondary importance.

The Author assured Mr Wolf that the Italian Regulations of 1931 were still in force. Generally, they prescribed for arch dams the analysis which considered the water load to be carried entirely by the horizontal arches, assumed as being independent from one another. However, they also admitted that the contribution of the cantilevers and of the solidarity of the rings could be relied upon whenever the morphological features and the chord-height ratio were favourable. On those reasonable bases, the application of the Regulations allowed in practice a remarkable development in the construction of arch-gravity and domed dams.

In an absolute sense, every curved dam was an arch-gravity dam; there were no definite structural differences between arch and arch-gravity types, and there were only two different conventional analytical procedures. According to the features of the location those two procedures might lead to more or less conservative dimensions. In some cases (for instance, and as Mr Wolf had argued, when the cantilevers were not considered as contributing to the strength) a certain increase in the thickness of the arches resulted, increasing in that way the general factor of safety. It should be borne in mind too that different methods of analysis might lead to substantially different results, according to the coefficients chosen.

The Author realized of course that some of the dams described in the Paper seemed remarkably slender; it seemed reasonable to assume that that slenderness could be increased, but not by very much at the present stage of technique. However, it had been ascertained from the breakdown tests carried out on the models that the safety factor was generally very high, as Mr Wolf had correctly assumed.

Substantially, an arched dam—still more a domed one—when well analysed (and eventually model-tested) and well built with suitable materials and in a good location, was a structure which possessed a remarkable surplus of strength. It might be added that " creep " of concrete contributed to centralization of the pressure curve, so that any increase of load produced mainly an increase in compressive stresses only.

The abutments, of course, had to be strong enough; however, the Author believed that in some cases it might be practicable to reconcile considerable differences of structure between the two abutments, by means of suitable structural methods. It had recently been proved by means of model tests that in some cases the arch could, in general, behave elastically even when lacking completely the upper part of the abutments for a height of about 30 feet.

Turning to the perimetral joint, the Author referred to the explanations already given by Professor Oberti. The history of the perimetral joint had begun with the Osiglietta dam (finished in 1939); it might be considered as a still open question, but in general the results had been excellent. The joint avoided substantially the arising of tension stresses in the area surrounding it; the result was an increase of compression on the downstream face. It should be noted that proper increments of thickness were always provided downstream, near the abutments.

Turning to the questions regarding large-size aggregate, the Author stated that, from every point of view, it was convenient to use crushed aggregates with sizes up to 120 mm. (about 4½ inches), also up to 150 mm. (about 6 inches) with gravel (always, of course, for thick structures). In that way the cement content could be reduced and workability was quite good, if proper vibrators were used. No special difficulty had been encountered. However, should the danger of frost be really serious, as for locations higher than 1,000 metres (about 3,300 feet) above sea level, it would be advisable not to exceed the size of 60 mm. (about 2¼ inches).

Mr Wolf's comments on the modulus of the bed rock had been dealt with in the reply to Mr Binnie. The test tunnels had been left under pressure sufficiently long for the maximum development of shrinkage and "creep," which in the local limestones and dolomitic rocks took place after only a few hours. The problem would be very different in clayey rocks, such as marls. The concrete lining of the test tunnels (which had an average thickness of 20 centimetres (8 inches)) had not been a continuous ring; it had been divided longitudinally by three joints and had been placed rather a long time before the tests.

Referring to Mr Wolf's suggestions regarding the type chosen for the Piave dam, the Author stated that numerous possible alternatives had been examined, but all had led to higher volumes of concrete. The Author was already aware of some of the examples given by Mr Wolf, but he thought that in those cases the "wings" were not so high as that of the Piave dam, which had to be 55 metres (180 feet).

The Author feared that he had not, in the Paper, clearly explained the actual situation, nor the full import of his deliberations on the problem. He had also designed a wing connected by one of its sides to the adjoining gravity-structure.

As a whole, the topographical conditions were indeed unfavourable. A straight gravity dam—which from a general point of view would have been the most economical—would have led, owing to the direction of the gorge, to vertical cross-sections like that shown in *Fig. 24*. As a consequence, it would have been necessary to cut off the rock wall down to the bottom of the gorge, and to replace it with concrete. Should the direction of the gorge have been parallel to the axis of the valley, no extra cost would probably have arisen for a straight dam.

From a general standpoint, higher tension stresses could be permitted in an arched structure than in a purely gravity structure.

However, in the sections reproduced in Fig. 8, Plate 2, the tension stresses existed only on the downstream face of some of them, in the case of an empty reservoir. The Author presumed that Mr Wolf's remark referred especially to the section at joint 28, corresponding to the plug. In such dams, it was always necessary to consider the structure spatially, not only through the two-dimensional elements. Should the dam be a gravity one, or should the plug be wider, the above-mentioned section would be subjected to prohibitive tension stresses; but the plug worked rather as a wedge (with internal arches, as mentioned in the Paper) and the arch gravity dam was simply supported by it. In other words, whilst the

Fig. 24

section of joint 28 was really striking, it could not be considered separately from the general plan of the plug.

Figs 10 were intended only to show some sketches indicative of the highest values recorded.

Generally all the joints tended to *close* at both downstream and upstream faces under maximum water-load. The left-hand graph in *Figs 10* corresponded to the joint between blocks 14 and 15, approximately in the middle of the dam. The points below the horizontal zero-lines indicated *closure*.

At the time when the concrete temperatures were registered (second diagram in *Figs 10*) the air temperatures were as follows :

With empty reservoir: 12° C.
After a rapid filling: 20–21° C.
With steady max. load: 2–3° C.

The water temperature might be assumed to be between 4° C. and 6° C. On the third graph, the lines of displacements gave the average values above or below the zero lines. In the following Table, + denoted movements *above* the zero line.

Elevation	Empty reservoir (beginning of May 1950)	After a rapid filling (end of May 1950)	Max. water load (end of December 1950)
630·00	+1 mm.	+8 mm.	+10 mm.
665·00	+2 „	−3 „	− 5 „
679·50	+1·5 „	−1 „	− 3 „

The bibliography suggested by Mr Wolf was being prepared and it would be published in later Papers (probably in "*L'Energia Elettrica*"), which would describe the dam in detail. A first list of publications was reported in the publication on the Italian dams edited by ANIDEL (Associazione Nazionale Imprese Distributrici di Energia Elettrica). However, the analyses of Professors Arredi and Oberti were mostly based on original studies, as stated in the Paper.

The Author assured Dr Jaeger that in the Lumiei dam the lines of the principal stresses were nearly horizontal, like the fissures in the model. The form and the chord-height ratio of the Val Gallina dam were quite different from those of the Lumiei dam, and that accounted for the differences in their behaviour. It was probable that if the model-tests for the Val Gallina dam had been extended to final breakdown the first fissures would have appeared in the central area; they would have been horizontal, because of the lines of the principal stresses being also horizontal in the same area. As Professor Oberti had stated also, the corresponding lines on the upstream face were, in general, nearly horizontal.

Professor Oberti had already replied to Dr Jaeger's comments on the analytical methods of Coyne.

So far as the Author was aware, discontinuous granulometry had never been used in Italy; and experience of air-entrainment products was not yet extensive. They had been used recently for the Mucone and Travignolo dams, but no results had yet been published.

Professor Baker's suggestion of the use of an elastic net for analysis had interested both the Author and Professor Oberti. It corresponded to a method already applied at the end of last century by a Catalan architect, Dr Gaudi, with the aim of determining the most suitable form for the domes of churches.

A proposal had been made at the Model Test Laboratory of the

Milan Polytechnic and at the same time by a Rumanian engineer for the study of the problem by means of membranes. However, many serious difficulties had to be overcome before that method could be applied to surfaces which could not be developed, owing to the shape of load funicu lars. On the whole, the Italian technique found it more convenient, for a constructional design, to start with the form of structure most suited to the characteristics of the location; then, as the results of model tests became available, every possible improvement could be made.

In conclusion, the Author emphasized that the problems involved in such constructions were extremely complicated and that even the most experienced designer should submit each assumption and each result to every possible test.

The problems in Italy were so complex and the conditions under which work had to be carried out were so different in each location that it was necessary to be very cautious in applying to any new job the procedure used for a previous one, even if the earlier results had been favourable.

Every problem should be submitted to the collaboration of several experts, since one opinion, though competent, might sometimes lead to unilateral results. (The Author's comments on intuition (p. 512) should also be borne in mind.)

The new Institute—the I.S.M.E.S.—which had been founded at Bergamo with the aim of developing the testing of large structures through models, would be a remarkable help in that direction.

The Author hoped that British engineers would have an opportunity to see the dams described and also the Institute at Bergamo, where any designer might propose the testing of a model and follow its progress.

Correspondence on the foregoing Paper is now closed, and no further contributions other than those already received at the Institution may be accepted.—SEC. I.C.E.

ORDINARY MEETING

22 April, 1952

ALLAN STEPHEN QUARTERMAINE, C.B.E., M.C., B.Sc.(Eng.), President, in the Chair

The President invited the Members present to stand while he read the following Resolution on the death of Professor Sir Charles Inglis, Past-President, which had been passed by the Council :—

"That the Council record the deep regret with which they have learned of the death of Professor Sir Charles Edward Inglis, O.B.E., M.A., LL.D., F.R.S., Past-President of The Institution, who was a member of the Institution for 50 years, and served on the Council for 18 years. He was elected a Vice-President in November 1938, and President in November 1941. The Council desire that an expression of their sincere sympathy be conveyed to the members of his family in their bereavement."

The Council reported that they had recently transferred to the class of

Members

William Henry Burren.
Ronald Brown Glover, M.C.
Alexander Allardyce Middleton, B.Sc. (*Edin.*).
Malcolm George Wilson.

and had admitted as

Students

Derek Allen.
Peter Hugh Allen.
Kathirippillai Annasingam.
Michael John Keith Bailey.
David Barclay.
John Wallace Bunce.
John Christie.
Edward James Collins.
Terence Coyne.
Alastair Thomas Currie.
Frank Colin Evans Davies.
Maxwell Remigius de Silva.
Bernard George Eagles.
Richard Hilary Edwards.
Derek Hurley Elton
David Ferguson.
Patrick Foody.
Geoffrey Arnold Foster.
Michael Dudley Hampson.
Thomas Hancock Harries.
Eric Keith Heads.
Richard Hegerty.
Douglas Hing.
Norman Howarth.
Peter James.
Thomas Michael Dennis James.
Ronald Vivian Jenkins.
Norman Keith Jones.
Thirugnanam Kathiravelupillai.
William Thomas King.
David William Lamb.
Paul Alfred Lamb.
Ian David Gordon Lee.
Mousa Sassoon Levy.
James Michael McCluskey.
Thomas Mann.
Richard Rawson Manning.
John Reginald Marsden.
Donald Edward Mitchell.
Leonard John Mitchell.
James Pringle Moffat.
Thomas Leyden Murphy.

Paul Clarke Nickalls.
John Blythe Parkinson.
Harry Orville Phelps.
David Allen Poole.
Derek Arthur Price.
Vernon Frederick Price.
John Purchase.
Reginald Frederick Ready.
Colin Eric Roper.
Alan Rushby.
Thomas William Seekings.
George Michael Senior.
John Alfred Shepherd.
John Skinner.
Derek Robin Stewart Smith.
John Lewis Spink.
John Douglas Storey.
Alexander Alfred Stuart.
John Brian Tattam.
Francis Tomnay.
Thambimuttu Vivekananthan.
Derrick Watson.
Alan George Weeks.
Colin Michael Whalley.
Vernon David Brinley Williams.
Robin Lee Wilson.
William Hector Charles Wright.

The following Paper was presented for discussion, and, on the motion of the President, the thanks of the Institution were accorded to the Author.

Paper No. 5857

"Sewerage and Sewage Disposal in Sub-tropical Countries, with Special Reference to South Africa and Mauritius"

by

Ernest John Hamlin, D.Sc., M.I.C.E.

SYNOPSIS

J. D. Watson, Past-President I.C.E., frequently said "There is no best method of sewage disposal which will meet all conditions." The Author has tried to show the differences between practice in Great Britain and in sub-tropical countries, and has stressed the importance of details in the design of sewage works for the latter. In Britain, great changes in the past 40 years have been the outcome of labour difficulties which have not been so apparent in the sub-tropics where, for example, mechanical screening and the dredging of detritus chambers are almost unknown. As a general policy, mechanization is avoided as much as possible.

The type of sewage to be treated depends a great deal upon the diet of the population concerned—a diet which ranges from one of the entirely carbo-hydrate intake of the Bantu to the emulsified blood and milk regimen of the Masai.

The African labourer is not a permanent resident in the cities and towns; he is frequently a carrier of poliomyelitis and a sufferer from intestinal affections caused by *Ascaris* (Round Worm). Special precautions are necessary to deal with the ova of these diseases.

The wide variation of rainfall—both quantitative and seasonal—adds to the difficulties involved.

The Paper deals with the historical development of sewage disposal in Southern Africa and gives details of special research work performed to meet local conditions. For instance, the production of cyanide (for the gold mining industry) from sewage gas, and the use of methane in the sugar and wine industries are discussed, together with the production of newsprint from bagasse—a waste product of the sugar industry.

Introduction

It is realized that the information in the Paper may not be of any direct assistance to Public Health Engineers practising in Great Britain, but it may help engineers elsewhere in the world, where conditions are similar to those pertaining in sub-tropical countries, particularly South Africa.

It must always be remembered that there is no *best* method of sewage disposal and anyone who slavishly copies a system which is a success in one city or town may find that it is not suitable for the locality he serves. Every sewage-disposal works must be designed to meet the particular conditions which exist in the area for which it is designed—there is no "rule-of-thumb" method.

Sewage may be defined as liquid containing varying amounts of organic and inorganic matter in solution and in suspension. This liquid consists of water, plus the solid and liquid excreta of the population, soap suds, etc. and the liquid and solid refuses from industry.

Table 1 gives the latest census figures for fifty towns in South Africa. These figures reveal important data regarding the distribution of population in the important urban centres of South Africa. In sub-tropical countries each section of the population has its own particular diet. The effect of diet on sewage disposal will be discussed later in the Paper; it is sufficient to say at this stage that the liquid and solid refuses from the human body vary with diet.

For the complete purification of sewage there are several distinct operations, and for each of these operations there is a choice between several alternatives. Local conditions regarding climate, type of sewage, topography, disposal of end products, supervision, availability of labour, and the site of the sewage disposal works in relation to residential areas must be considered. It should be obvious, therefore, that there cannot be—as many appear to think—any particular method of purification which is universally applicable to every type of sewage.

The question of management is most important; a good manager will often produce a better effluent from an old-fashioned works, than a bad manager from the most expensive and up-to-date works. To pay a good manager adequately is an excellent investment.

The Object of Sewage Disposal

The main object of sewage disposal is the production of:

(*a*) clean water;

(*b*) sludge suitable to be used as a fertilizer;

and (*c*) gas;

with due regard to economy.

In a country where the general population does not know a great deal about sewage disposal and where there are very few works which can be visited, there are prejudices which are very difficult to overcome. There are many towns using a bucket removal system to drain all their slop water, bath water, etc., to a river, which is always quite adequate to purify this waste water. The lower riparian owner rarely objects to it being discharged into the river, because he uses the effluent for irrigation. If, however, an attempt is made to instal a complete sewerage and sewage disposal scheme, and drain even a chlorinated effluent from the sewage works into the river, then great opposition is encountered. It is difficult to understand this attitude of mind.

It is scarcely necessary to remind Public Health Engineers that the administration of public health, insofar as it concerns measures safeguarding the health of the people, is always a compromise between the ideal and the

attainable; but it is not so fully appreciated that it is often essential to weigh the advantages and the good, likely to be achieved, against any harm which may be caused by an interplay with other factors having bearing on the public weal. Another factor, which is sometimes overlooked, is that a theoretical danger or theoretical advantage may have been largely, and quite often entirely, nullified in practice. Also, in the complex environment of modern urban life the legislator and the administrator of public health measures have to be prepared to take what the military tactician would call "calculated risks"; but there is a very wide gulf between "foolhardy taking of chances" and a "calculated risk."

The Author feels that all will agree that what is done with the end products of the ever-increasing sewage treatment works is of considerable importance from two points of view. First, these products may become a danger to health, and secondly, their value in food production. Medical officers and engineers have to be confident in their conclusions and advice, and to be sure that they strike a wise balance between these two factors. It is a very grave responsibility which has to be discharged and these technical advisers realize this and will not allow any preconceived notions or false enthusiasms to sway them. The great and growing importance of food production should be constantly borne in mind, particularly in a country like South Africa where, since it is a dry country and much of its soil is poor, every gallon of water and every pound of fertilizer is precious.

It must be borne in mind that perfection is virtually unattainable. The advantages have to be carefully weighed against the disadvantages, and the authorities, whilst giving full consideration to practical experience, should be prepared to take wisely-calculated risks and must not allow their minds to be led astray by theories.

Short History of the Development of Sewage Disposal Works in South Africa

The development of sewage disposal in South Africa falls into three distinct periods: (*a*) 1898 to 1915, (*b*) 1916 to 1935, (*c*) 1936 to the present day. In the early stages it was naturally greatly influenced by British scientists and engineers.

Period 1898 to 1915

In the earlier years of this period, the work of the late Donald Cameron of Exeter was embodied in the design of most of the sewage works constructed.

The works at Wynberg, Cape Colony (1898), at Bloemfontein (1901), and at Johannesburg (1906) were all designed on the use of septic tanks. At the two former works, percolating filters were incorporated before irrigation over land, but Johannesburg used the effluent direct from the septic tanks for irrigation.

The Pietermaritzburg Sewage Disposal Works were the first in South Africa to have sedimentation tanks of the hopper-bottomed type substituted for septic tanks. The designer, Mr F. Walton Jameson, must have been influenced by the work of the late Mr O'Shaughnessy (Birmingham) and introduced separate sludge digestion. If he had not been appointed City Engineer of Pretoria and had had time to control the operation of the Pietermaritzburg works, an amount of valuable information would have been obtained.

The works at Pietermaritzburg were outstanding—they were amongst the first complete works in the World designed with separate sludge digestion. It is very difficult to realize that such well-designed works were allowed to become derelict within a few years.

In 1910 Mr Jameson designed the works in Pretoria on the same principles as those of Pietermaritzburg, namely, screens, detritus chambers, sedimentation tanks, percolating filters plus separate sludge digestion. At that time, the value of humus tanks and sand filters was not realized; however, by splendid management the works continued for nearly 30 years to be amongst the best in South Africa. Pretoria was the only local authority allowed to discharge its effluent into a stream until, in 1928, a similar privilege was granted to Johannesburg for its "Antea" sewage disposal works.

During the period under review, sea outfalls were built in Cape Town, Green and Sea Point, Muizenberg and Kalk Bay, and Port Elizabeth to discharge crude sewage into the sea at all periods of the tide. At Durban retaining tanks were constructed and discharge took place at ebb tides only.

Period 1916 to 1935

Three very important factors affected sewage disposal during the period 1916 to 1935.

(1) The passing of the Union of South Africa Act, which granted independent status to South Africa. Instead of having four Public Health Departments, one for each of the four colonies (Cape, Natal, Orange Free State, and Transvaal), a Public Health Department for the whole of the Union was established.
(2) The passing of the Union Health Act, No. 36, 1919.
(3) The 1914–18 war.

In addition to the Union Parliament, each Province had its own Provincial Council. These Councils performed similar functions to those performed by the Ministry of Health in Great Britain, but they were responsible, in addition, for education up to matriculation standard and town planning.

A local authority wishing to raise capital for the construction of a sewerage and sewage disposal scheme first had to obtain the approval of the Provincial Administration of the Province in which it was situated.

In order to protect local authorities most of the Provincial Administrations submitted any proposals for sewage disposal works to the Chief Health Officer of the Union. His consent usually embodied a condition that the local authority must, in addition to having a complete treatment plant, provide 100 acres for every million gallons per day to be treated. It is very difficult to understand why such a condition should have been imposed. One million gallons is equal to 3·673 acre-feet of water. Therefore, if the average flow is one million gallons a day, it means that 1340·65 acre-feet of water must be irrigated on 100 acres of land every year. This is equivalent to 161 inches of irrigation water per year in addition to the rainfall. Such a quantity of water cannot be absorbed by the land and thus one of two things must result :—

(1) There is a large run-off from the land into the adjoining streams, or

(2) the water-table in the neighbourhood must be raised.

In a case fought in the Supreme and Apellate Division of South Africa it was proved that the local authority had complied with the conditions laid down by the Public Health Department, namely, providing 100 acres of land, but by applying this quantity of treated sewage, it had raised the water-table of the neighbourhood, which affected the wells in the surrounding district. Although it was proved that the water entering the wells was bacteriologically pure, it did contain a high percentage of chlorides which led the judge to agree that the water must have come from the neighbouring sewage disposal works and was therefore interfering with the rights of the owner of the well.

Attention is called to Section 132 (1) (e) of the South African Public Health Act, which reads :—

> " The Minister may make regulations, and may confer powers and impose duties in connection with the carrying out and enforcement thereof on local authorities, magistrates, owners and others, as to :—
>
> the standard or standards of purity of any liquid which, after the treatment of sewage or other offensive matter in any purification works, may be discharged therefrom as effluent."

The result of the above-mentioned case was that the local authority was ordered to cut trenches all round its disposal site to a depth of approximately 6 feet to intercept ground water and discharge the resulting effluent into the river. In other words, they were ordered by the Supreme Court to do something for which permission could not be obtained from the Chief Health Officer of the Union.

As a matter of fact, from the time of the passing of the Act about 32 years ago, the Minister has not exercised his powers under Section 132 (1) (e), and in only three cases has permission been granted for effluent to be discharged, after passing through sand filters, direct into the stream.

However, standards have now been proposed and it is hoped that they will be approved by the Minister. These standards are :—

(1) *Colour, Odour, and Taste.*—A snap sample of the effluent shall contain no substances producing, or capable of producing, colour, odour, or taste in such concentrations that they cannot be readily removed by the normally used processes of water purification (for example, aeration, flocculation, etc.).

(2) *Settleable Solids.*—A snap sample of the effluent shall contain not more than 0·5 millilitre of settleable solids per litre of effluent (2·4).

(3) *Suspended Solids.*—The effluent shall contain not more than 40 parts per million of suspended solids in any snap sample, and not more than 30 parts per million in an average sample.

(4) *Dissolved Solids.*—The increase in the concentration of dissolved solids, in an average sample of effluent over and above the concentration of dissolved solids of the normal water supply of the district, shall be within such a limit that the subsequent users of the water into which the effluent discharges shall not be prejudiced.

(5) *Oxygen Absorbed from N/80 Permanganate in 4 Hours at 27° C.*—The effluent shall absorb not more than 20 parts per million of oxygen in any snap sample, nor more than 15 parts per million in an average sample.

(6) *Relative Stability.*—When subjected to the methylene blue test, a snap sample of the effluent shall have a minimum stability of 90 per cent.

(7) *Dissolved Oxygen.*—A snap sample of the effluent shall contain dissolved oxygen to the extent of at least 50-per-cent saturation.

(8) *Hydrogen.*—The *pH* value of a snap sample of the effluent shall be between 5·5 and 9·0.

(9) *Free Chlorine.*—An average sample of the effluent shall contain no free chlorine.

(10) *Pathogenic Organisms and Viable Parasitic Ova and Cysts.*—A sample of the effluent shall be free from organisms pathogenic to man or to slaughter-animals used for human consumption, and from viable parasitic ova and cysts.

(11) *Soap, Oil, Grease and Soluble Oils.*—A snap sample of the effluent shall contain no soap, oil, grease, or soluble oils.

(12) *Temperature.*—The temperature of a snap sample of the effluent shall not exceed 45° C.

(13) *Other Constituents.*—A snap sample of the effluent shall contain not more than the stated concentrations of the following constituents :

(a) *Free ammonia* (*as N*) . .	25	parts per million
(b) *Arsenic* (*as As*) . . .	2·5	,, ,, ,,
(c) *Chromium* (*as Cr*) . .	1·0	,, ,, ,,
(d) *Copper* (*as Cu*) . . .	1·0	,, ,, ,,
(e) *Cyanide and related compounds* (*as CN*)	2·5	,, ,, ,,
(f) *Phenolic compounds* (*as phenol*)	0·01	,, ,, ,,
(g) *Lead* (*as Pb*) . . .	1·0	,, ,, ,,
(h) *Sulphides* (*as S*) . . .	1·0	,, ,, ,,

About 2 years ago the South African Government appointed a Judicial Commission to make recommendations regarding the alteration of the Water Laws in South Africa. It would appear, from statements made by responsible Ministers of State that, at a not too distant date, effluents from sewage disposal works, with reasonable safeguards imposed, will be allowed to be discharged direct into a watercourse, or be used for irrigation purposes by private individuals instead of being wasted by local authorities.

Naturally there are difficulties, because very few streams or rivers in South Africa run all the year round. The late George Bernard Shaw, in one of his typical phrases, once said: "If you fall into a South African river, be sure you have your clothes brush with you."

From records available it has been established that between 60 and 70 per cent of the water sold by a local authority is returned via the sewers to the sewage-disposal works.

Recently the Union Public Health Department, realizing that its policy of forcing local authorities to purchase 100 acres for each million gallons of sewage to be treated per day was not sound, has been encouraging local authorities to buy 400 acres for each million gallons to be treated. This would appear to be economically unsound, and difficult to justify.

At the outbreak of World War 1, a great number of industries were established in South Africa. With the introduction of these industries came the problem of dealing with industrial effluents, and this is dealt with later in the Paper.

Between 1916 and 1935 there was great activity regarding the installation of sewerage and sewage disposal in South Africa. Only one large coastal town was without sewerage and sewage disposal, namely, East London, but that position was rectified during this period.

It is true that in Capetown the greater portion of its area had not been sewered, but it was only a few years before this period that the present area of Capetown came under one jurisdiction. Previously there had been twelve separate local authorities. The late Mr Lloyd Davies was appointed City Engineer of Capetown, and one of the first major works undertaken by him was the sewerage and sewage disposal of the whole of the new Capetown area. In his original report he recommended, very definitely, that all sewage should be discharged into the sea, but opposition was so great

that it was abandoned, and outfall works were constructed to serve the whole of the southern suburbs of Capetown. This was put into operation in 1921. All the sewage had to be pumped, and the method of treatment was:

(1) screens;
(2) sedimentation tanks of the Imhoff type;
(3) oxygenation of the tank effluent on brushwood percolating filters; and
(4) humus tanks followed by irrigation on land.

In the Imhoff tanks the period of sedimentation was approximately 1½ hours and the sludge digestion chamber was designed to hold about 6 months' supply of sludge. The digested sludge was dried on special beds and the dried sludge was used as a fertilizer on the sandy area of the Cape Flats. The irrigable area was more than 1,000 acres in extent and the drainage of this area eventually reached the Black River and was discharged directly into Table Bay.

In 1917 the first small town in South Africa to have sewerage and sewage disposal, namely Queenstown, had works to deal with about 400,000 gallons of sewage per day. The installation was septic tanks in duplicate, followed by percolating filters and the discharge of effluent on to 100 acres of land.

Immediately after the conclusion of World War 1, Mr F. Walton Jameson was appointed City Engineer of Kimberley. He had been City Engineer of both Pietermaritzburg and Pretoria; he had designed sewage-disposal works for both these cities and had little difficulty in persuading the city of Kimberley that it was in its interest to have sewage disposal in that city. A scheme to serve the central area of Kimberley was designed and the works were constructed in 1923. The works were very simple in design, being composed of (1) screens; (2) sedimentation in modified Dortmund type of tank; and (3) broad irrigation of the land by tank effluent.

Kimberley has a low rainfall, very high evaporation—about 90 inches a year—and the site selected was very suitable.

The sludge from the sedimentation tanks was taken directly on to land. The method adopted by Jameson was to have ten undrained areas, each measuring 20 feet by 200 feet, that is, a total area of approximately 1 acre. These ten beds were used in strict rotation, and fly breeding was prevented by continuous harrowing.

The next scheme to be constructed was that at Stellenbosch, which was designed by the Author, to deal with half a million gallons per day from a population of approximately 10,000. The treatment plant consisted of screens and square sedimentation tanks—on the Dortmund principle, but with an upward flow instead of a downward flow as at Maritzburg, Pretoria, and Kimberley. The sedimentation-tank effluent was treated on four 100-foot percolating filters and then discharged on to approximately 200 acres of land; several small dams were constructed to hold the drainage from

this area, the capacity being about 14 days' flow. Separate sludge digestion was installed.

During the period under review a tremendous amount of research work was done in South Africa under the guidance of Mr D. E. Lloyd Davies of Capetown, Mr Croghan, the Bio-Chemist of the City Council of Capetown, Mr F. Walton Jameson, Mr Marshall Lundie, the Bio-Chemist at Pretoria, Dr Wilson, and the Author. The Author had been appointed Assistant City Engineer to deal specifically with sewerage and sewage disposal problems in Johannesburg. The Johannesburg City Council established a central Bio-Chemical Laboratory and Dr Wilson was appointed to control the activities of that institution.

Problems of vital importance in sewage disposal so far as South African conditions were concerned were investigated. These investigations were carried out according to a definite plan and have proved of inestimable value to the whole of South Africa.

During the latter half of the period under review more than a dozen new sewage disposal plants were constructed. In Johannesburg alone four new plants were designed and constructed and a beginning was made on a re-design of the original works at Klipspruit.

The first works designed were to deal with domestic sewage and industrial waters—the latter being the greater part of the flow. They were completed early in 1930. The designed capacity of the works was half a million gallons per day and they were the first works in South Africa to use re-circulation and sand filters.

These works were in operation until recently and treated more than a million gallons per day. The effluent was discharged directly into a stream and was, until recently, used by one of the gold mines in the neighbourhood.

Period 1936 to Present Date

Early in 1936 several sewage disposal works in South Africa were in the design or early construction period. Probably the most important of these was the one established to deal with the sewage of Germiston and Boksburg. The screening chamber, grit channels, sedimentation tanks, and pump sumps were housed in a large hall similar to that which had been installed on the Delta works in Johannesburg. A Prüss filter, totally enclosed, was constructed alongside an open 6-foot-deep filter for comparative tests. The sewage from this area is very concentrated and contains a larger proportion of strong industrial waste.

Dr K. A. Murray was appointed Bio-Chemist and conducted research investigations on two-stage filtration.

At Springs a similar plant to that built at Germiston was installed in 1936 with totally enclosed artificially ventilated filters. However, after thorough investigation by the Bio-Chemist and as a result of the research work done by Dr Meyling at Johannesburg, open 12-foot-deep filters were adopted for the extension of these works. Sand filters were also introduced

and the effluent from the works used as cooling water for an industrial plant nearby.

As a result of the work done at Johannesburg, Capetown, and Pretoria the Municipalities of Brakpan, Benoni, and Krugersdorp all carried out sewage disposal works embodying special features to meet South African conditions. Further, many other local authorities within South Africa established works at this period. Reconstruction of all main works was started at Capetown, Johannesburg, and Pretoria.

During the early part of the war, plants were installed by the Defence Authorities for military camps, hospitals, and aerodromes throughout the country. Some of these plants were designed to deal with the sewage from a population as high as 100,000, as was the case in a prisoner-of-war camp near Pretoria. Most of these plants were designed either by the Public Works Department of the Union or by specialist firms, and many of them incorporated totally enclosed filters of the Prüss type, mechanical clarifiers or clarigestors, two-stage bio-filters, and aeration units.

One notable feature was that at one prisoner-of-war camp chemical precipitation was practised. The cost was almost prohibitive so far as chemicals were concerned, but it was justifiable under the war conditions pertaining at that period. It was then known that at the end of hostilities the whole plant would have to be abandoned, and although the chemicals were expensive, the total cost was not nearly so great as the capital which would have been invested to install a modern plant.

During the same period sewage disposal works were installed at a great number of mines where the local authority could not give service, and today the most up-to-date plants and processes are being utilized, especially designed to meet South African conditions.

The policy adopted by the Johannesburg City Council, resulting from research and investigations, caused a general awakening throughout the country to the scientific viewpoint on the subject of sewage disposal, and it provided an example which the Government and higher authoritative bodies have unfortunately failed to emulate.

Unfortunately legislation has not kept up-to-date with research work; this has unnecessarily handicapped the science of sewage disposal and has complicated in no uncertain way the position that has arisen regarding the question of stream pollution. It is a great pity that in South Africa, where the need of water for irrigation particularly is desperate, the Government has not given this question greater consideration. When it is remembered that between 60 and 70 per cent of the water sold by any local authority that has sewage disposal is returned by way of the sewers to the works, its re-use is not only a necessity but is essential to the economics of the country.

It is hoped that the recently appointed Judicial Commission will recommend that greater use of sewage effluent be made. The Irrigation Department of the Union is aware of the value of such water and, some years ago,

was prepared to use as much as 10 million gallons a day from one of the works to be constructed in Johannesburg.

GOVERNMENT CONTROL OF SEWAGE DISPOSAL WORKS

The operation of sewage disposal works in South Africa is controlled by the Chief Factory Inspector, and the Factories Act is rigidly applied. This factor must be considered. For instance :—

(*a*) All employees work no more than 44 hours a week.

(*b*) The rate of pay is determined by law except for those employees earning £540 or more, exclusive of cost-of-living allowance, per annum.

(*c*) If mechanical plant is used, where either any single unit of 75 H.P. or more, or 250 H.P. in combined units are installed, a Government certificated engineer *must* be appointed. The conditions of the award of the Government certificate are that a full apprenticeship must have been served and an examination passed. Very few civil engineers can fulfil these conditions and so must delegate their responsibilities, which is not satisfactory.

The conditions have an influence on the design of the sewage disposal works from an economic point of view.

In addition, the Public Health Authorities have an interest in the design and operation of all sewage disposal works. In this connexion it may not be out of place to quote a section of the South African Public Health Act. Section 143 reads :

"Whenever in the exercise of any power conferred or in the performance of any duties imposed upon the Government or any officer thereof or an administrator or a local authority under this Act or any other law relating to public health, he or it is alleged to have caused injury to any person or damage to any property or otherwise to have detrimentally affected the rights of any person, whether in respect of property or otherwise, it shall be a defence in any legal proceedings *founded on such an allegation and brought against the Government or its officer or the administrator or a local authority or its officer that the defendant or respondent has used the best known or the only or most practicable and available methods in the exercise of the powers or the performance of the duties aforesaid.*

"In the case of such proceedings against an administrator or a local authority a certificate signed by the chief health officer that the defendant or respondent has, when regard is had to all the circumstances, used the best known or the only or most practicable and available methods, shall be accepted by the court as prima facie evidence of the fact."

This affords the local authority protection, but imposes a very heavy responsibility on the Chief Health Officer of the Union.

Availability of Labour and its Effect on the Design of Sewage Disposal Works

During the last two decades the design of sewage disposal works in Great Britain has been influenced by the inability of local authorities to obtain the type of labour which was formerly available. Modern sewage disposal works in Great Britain are being rapidly mechanized.

In most sub-tropical countries unskilled labour is available in quantity, which results in mechanization being reduced to the minimum. Further, the cost of unskilled labour in Africa and the islands off its shores is, on an average, 30 per cent of that in Great Britain.

Most of the machinery installed on sewage disposal works is made in Europe or America. When installed in South Africa it costs from 80 to 100 per cent more than in Great Britain, the main reason being that in order to operate this machinery it is necessary to carry a full range of spares, which is not usually the case when the installation is in the country of manufacture.

From what is stated above it may be seen that it will very rarely be economical to install machinery for such activities as screening and detritus-removal.

Movement of Labour and its Effect on Sewerage and Sewage Disposal

To those who are accustomed to European conditions it is very difficult to understand the conditions in Southern Africa, which includes the Union of South Africa, the Rhodesias, and South-West Africa.

In round figures the population of the Union of South Africa is between 11½ and 12 millions, made up as follows :—7¾ million Africans, 2½ million Europeans, 1 million coloured persons of mixed European and non-European origin, and one third of a million Indian and other Asiatics.

It may not be out of place to state that the native population of the Union of South Africa migrated to the Union only during the seventeenth century—about the same time as the first European settlers came to the Cape Colony. There are many who believe that the African is indigenous to Southern Africa.

During the last 30 years South Africa has undergone an industrial revolution, during which period the number of all races working in industry (excluding mining) has increased six-fold. However, the greater portion of this section of the population is migratory in character. Very few Africans working in industry, for local authorities or for the mines, regard themselves as permanent urban residents.

Africans obtaining work in the gold-mining industry stay for an average period of 13½ months, and then go back to the Native Reserves for 7½

months. The African labour employed by the Johannesburg City Council has a complete turnover in 18 months, the average time they return to the Reserves is approximately 6 months.

The whole question of migratory labour has been admirably summed up in the " African Factory Worker " published by the Oxford University Press.

" Undoubtedly, there exists among the Bantu, just as among other peoples, the world-wide phenomenon known as ' the drift to the towns.' But parallel with this, there exists a curious but understandable reluctance to sever finally all ties with the tribal life in the reserves and locations and on the farms . . .

" An African in the country, with his kraal, his wife, some cattle, and with daughters growing up who will, on marriage, bring more cattle to him; such a man regards himself as an ' umnumzana '—a person of substance. He works in town, it is true, but at the end of ten months or a year, if he does not squander, he goes home with enough money in his pocket to live a life of lordly ease for the greater part of another year before he need bestir himself to seek a job in town. Even if he squanders the lot, he can still live for half a year or so on the produce of his lands, the milk from his cows and the flesh of his goats.

" Cut him off from all this and put him to reside in town, without land, without cattle; with only his job, and probably a temporary one at that, to support himself, his wife, and his family. ' Aku'muntuwaluto ' says the African of such a one, ' he is a man of nothing '; in short he is nobody."

This method of living has a distinct bearing on methods of sewage disposal in South Africa. In the Reserves very unsanitary conditions usually pertain and when the African returns to work in the cities and towns he is usually infested with intestinal parasites, the ova of which must be intercepted by sand filtration and passed to sludge digestion tanks in which they have to undergo a longer period of digestion—as much as four times as long as is usual in Europe or America.

Disease and the Sewage Disposal Problem

Southern Africa, in common with many sub-tropical countries, does not appreciate the tremendous benefit derived by a community from the installation of modern sewage disposal works. It is amazing to realize that 95 per cent of the cases of blindness in South Africa could have been avoided by proper systems of sanitation and hygiene.

It is realized that engineers in Great Britain are familiar with the benefits of proper sanitation so far as typhoid fever is concerned, but the Author would like to mention three of the diseases not so well known in Great Britain, but which are prevalent in sub-tropical countries :—

(*1*) *Poliomyelitis* (*Infantile Paralysis*)

Dr E. H. Cluver, Director of the South African Medical Research Institute, in his Presidential Address at the sixth congress of the Health Officials' Association of South Africa stated :—

> " Repeated observations by workers in Johannesburg and other parts of the World have established the fact that the virus leaves the bodies of infected persons in large quantities in the stools. It is readily recovered from the stool of patients and from those of abortive cases and contacts. This is easily explained by the escape of the germs from the lining layers of the upper portions of the alimentary canal into the lumen of the large intestine. During epidemic periods in Johannesburg, Dr. Gear of the South African Institute of Medical Research found the virus in the sewage entering the Cydna Sewage Works. The infection was traced through the plant, from the point where it entered, in the settled sewage, in the raw sludge, and in the effluent from the humus tank ; *it was not found in the sand filtered effluent.* He found it also in the sewage works at times when no cases of poliomyelitis were present in the community served by the works. At such times the virus was presumably derived from silent cases or carriers."

At a conference held in Pretoria on " Sewage Sludge as a Fertilizer," Dr Gear, who is the President of the International Commission on Poliomyelitis, and Dr Veronica Measrock stated :—

> " It will be worthwhile considering this question in some detail, noting especially whether there is reason to suspect sewage disposal plants of being concerned in any way in spreading the infection. The virus of poliomyelitis is found in the throats of patients for 3–4 days before and for 4–5 days after the onset of symptoms. This is the period when patients are most infectious. It is also found in the faeces of patients regularly for 3–4 weeks, often for 8 weeks, and occasionally for as long as 12 weeks after the onset of symptoms."

In the course of a systematic study of poliomyelitis, an investigation into the presence, persistence, and distribution of virus in a sewage purification plant was undertaken with the co-operation of the Public Health and City Engineer's Department of the Municipality of Johannesburg. It obviously was important to determine whether the virus was present in the sewage of an infected area and, if so, whether it survived the treatment of sewage purification. Accordingly, by arrangement with Dr McLachlan, Officer-in-Charge of the Bio-Chemical Laboratory, Johannesburg Municipality, and his predecessor Dr Wilson, the required specimens were collected from the Cydna Sewage Farm. In this plant the sewage is fully

treated. It first passes through coarse screens and then into settlement tanks. The outflow from the settlement tank flows by gravity on to sprinkler filter beds. The effluent from these filters then passes through humus tanks on to fine sand filters. The final effluent from these sand filters flows on to irrigate cultivated lands.

The first specimens of settled sewage and the final sand filter effluent were collected on the 23rd June, 1946. This was about 1 month after the last known case of a small outbreak of infantile paralysis in the area had been removed to the Johannesburg Hospital. The virus was proved to be in the settled sewage, *but was not detected in the effluent from the sand filters.*

On the 19th and 20th February, 1946, a more detailed examination was undertaken to determine the distribution of the virus during the process of sewage purification. The specimens collected included settled sewage, raw sludge, treated or digested sludge, the effluent from the humus tank, the effluent from the sand filter beds, sewage (psychoda) flies, and faecal droppings from European swallows resting on telephone wires in the grounds of the sewage farm. The results of the tests for the presence of poliomyelitis virus were as follows :—

Specimen	*Presence of virus of poliomyelitis*
Raw sludge . . .	+
Digested sludge (30 days) .	—
Settled-sewage . . .	+
Humus-tank effluent .	+
Sand filter effluent . .	—
Psychoda flies . . .	—
Swallow droppings . .	—

This investigation thus revealed that the virus was still present in the sewage nearly 2 months after the last known case of poliomyelitis had occurred in the area, and indicates fairly clearly that, although no fresh clinical cases had been recognized during this time, a large number of silent infections were still in the suburbs served. It is noteworthy that the virus was present in the humus-tank effluent, which is the final effluent from many sewage works. Such an effluent, therefore, may contaminate streams with the virus of poliomyelitis.

The virus was not detected in the final and filter effluent, but further examinations were necessary before it could be concluded that this was a constant finding. Virus was not detected in the specimen of settled sewage collected in March 1946.

Since September 1946, specimens of settled sewage have been collected and examined at monthly, and more recently, at fortnightly intervals. In addition several specimens of the final sand-filter effluent have also been tested. The virus was not detected again until the 5th January, 1948,

when an outbreak had already assumed epidemic proportions, and the results of the tests were as follows :—

Date (1948)	*Settled sewage*	*Final sand-filter effluent*
Jan. 5th	+	−
Jan. 19th	+	−
Feb. 6th	+	−
Feb. 17th	+	−
Mar. 2nd	+	−
Mar. 16th	+	−
Mar. 31st	+	−
Apr. 12th	−	−
Apr. 28th	−	−
May 7th	+	−

On all occasions when the settled sewage was positive, the final sand-filter effluent was negative. It should be made quite clear that the tests had limitations, and small quantities of the virus might not have been detected.

Other methods of sewage disposal were investigated. Brief mention will be made of the findings.

By arrangement with Dr N. L. Murray, recently Medical Officer, Transvaal Provincial Peri-Urban Areas Health Board, the effluent from two composting works were examined. One, which he regarded as satisfactory, was negative. The other, which at the time he regarded as unsatisfactory, was shown to contain virus of poliomyelitis.

The effluent from a septic tank serving a country hotel, in which one case of infantile paralysis had occurred, was tested two months after this case had been removed to hospital, and was found to contain virus.

By arrangement with Dr A. J. Orenstein, the effluent from the sewage works of one of the Rand mines was examined. Virus was not detected.

The question arises as to whether the presence of the virus of poliomyelitis in sewage has any importance in the spread of the infection. Theoretically, of course, it may. The finding in the humus-tank effluent of the Cydna works probably means that the virus is present in the final effluent of most sewage works during an epidemic, and may thus contaminate the rivers and streams into which they flow, and in turn may contaminate water supplies. As the effluents from sewage plants are often used to irrigate vegetable gardens, the products of these may be contaminated by the virus. Then flies, other insects, and arthropods, may feed on the sewage and so become infected and later contaminate food and milk.

These theoretical possibilities should be examined in the light of the 1948 epidemic. First, there was no early concentration of cases in the neighbourhood of sewage works, as would have been expected, if they had been an important source of infection. Secondly, there was no correlation at all between the distribution of cases and the distribution of the water supply. Such a correlation has been noted in some epidemics in Sweden and the United States. Thirdly, on a number of occasions vegetables, including

tomatoes, lettuce, carrots, marrows, and fruits from plants and trees irrigated with humus tank effluent were examined and on no occasion was the virus detected.

So it seems that there was little evidence in the recent epidemic to incriminate properly designed and supervised purification plants in the spread of poliomyelitis.

(2) *Ankylostiniasis* (*Hookworm*)

During the Author's discussions with the Chief Medical Officer of Mauritius he was informed that the incidence of Hookworm had assumed such proportions as to constitute a menace to public health. It was prevalent throughout the whole Island but in the sewered area it was only 30 per cent of that found in other areas.

(3) *Ascaris* (*Round Worm*)

Most Africans from the Reserves are carriers of this disease. In Mauritius a surgeon told the Author that he had removed 5½ pounds of round worms from a child of 7 years of age.

General

From the experimental work and practical experience, the Author believes that, by the use of sand filters and properly designed sludge-digestion tanks, it is possible to prevent or greatly reduce the incidence of diseases caused by intestinal parasites. Sand filters, especially, have been found to be of inestimable value.

Diet and the Sewage Disposal Problems

The industrial waste from any factory depends upon the raw material used and the processes of manufacture.

It is generally recognized that the waste products from a food factory are difficult to treat. The human body should be looked upon as a food factory: therefore the waste from the human body depends upon the food eaten, and the quantity of waste materials depends, in no small measure, on the quantity of food consumed.

The African does most of the labouring work in Southern Africa, and his diet is principally carbohydrate in character—crushed mealies or Indian corn. This has a very low protein content and to do a satisfactory day's work it is necessary for him to consume a large quantity of this type of food—much greater in weight than is consumed by the European, who lives on a more concentrated diet. The diet of the Masai—the warrior of East Africa—is chiefly composed of emulsified blood and milk. He extracts blood from his oxen and milk from his cows. Some medical authorities say that this is a perfect diet, but it is far more concentrated than the European diet.

When many of the sewage disposal works in South Africa were designed,

the designer had to follow established practice, but it soon became evident that established practice was not applicable to South African conditions. It was found that the gas yield per head of population was greater than average but the sludge was not completed digested.

After experimental work at various sewage disposal works it was found that the amount of sludge-digestion capacity required varied considerably.

The Author had long discussions with the Johannesburg City Council's Bio-Chemist and Dr Fox—a specialist on diet—at the South African Institute of Medical Research. It was decided that Dr A. R. P. Walker should endeavour to determine the influence which the African population had on the capacity required for sludge digestion. Dr Walker had hardly commenced his investigation in the City Council's laboratories before he joined the staff of the South African Institute of Medical Research to work upon the question of diet and the resultant waste products discharged from the body.

After some years, records were obtained which can be stated briefly as follows :—

(1) *European concentrated diet*
- (*a*) 20–30 grammes of dry faecal matter per day.
- (*b*) 2–4 grammes of fat per day.

(2) *European brown-bread diet*
- (*a*) 40–53 grammes of dry faecal matter per day.
- (*b*) 2–4 grammes of fat per day.

(3) *African carbohydrate diet*
- (*a*) 60–100 grammes of dry faecal matter per day.
- (*b*) 6–15 grammes of fat per day.

From the information obtained from Dr Walker's research work it will be seen that when designing sludge digestion tanks it is essential to take into account the waste products from different classes of population draining to the sewage disposal works.

Speaking generally it is now usual in South Africa to provide a sludge capacity of 2·5 cubic feet per European and 5 cubic feet per non-European, instead of 2·5 cubic feet capacity as was done formerly. Even 2·5 cubic feet per capita is high compared with British practice, but it is essential in South Africa to have longer periods of digestion to destroy the ova of many intestinal parasites.

From Table 1 and using the figures given above :—

8	per cent of the towns require	4·5	cubic feet per capita.
16	,, ,, ,, ,, ,,	4·25	,, ,, ,, ,,
48	,, ,, ,, ,, ,,	4·0	,, ,, ,, ,,
16	,, ,, ,, ,, ,,	3·75	,, ,, ,, ,,
6	,, ,, ,, ,, ,,	3·5	,, ,, ,, ,,
4	,, ,, ,, ,, ,,	3·0	,, ,, ,, ,,
2	,, ,, ,, ,, ,,	2·5	,, ,, ,, ,,

TABLE 1.—PRELIMINARY POPULATION FIGURES FOR 1951 CENSUS OF FIFTY URBAN CENTRES IN SOUTH AFRICA.

Town	Europeans	Asiatics	Cape Malays	All other colour-eds	Natives	All races	Percentage of Europeans to total population
Johannesburg	326,150	17,051	3,935	28,775	430,372	806,283	40·45
Cape Town	242,493	7,740	45,860	214,580	51,180	561,853	43·16
Pretoria	149,614	5,603	443	5,445	119,670	280,775	53·29
Durban	148,980	157,951	798	15,024	143,863	466,616	31·93
Port Elizabeth	78,315	4,059	2,836	370,082	64,779	187,071	41·86
Germiston	65,854	1,796	34	2,050	80,801	150,535	43·75
Bloemfontein	47,856	3	20	3,422	56,181	107,482	44·52
East London	43,668	1,495	183	5,905	39,444	90,695	48·15
Maritzburg	31,512	15,936	147	3,212	22,086	72,893	43·23
Springs	31,389	1,014	41	905	82,531	115,880	27·09
Brakpan	29,463	47	35	346	54,391	84,282	34·96
Roodepoort-Maraisburg	29,217	738	30	1,591	46,400	77,976	37·47
Benoni	28,082	1,386	188	3,282	60,500	93,438	30·05
Krugersdorp	26,370	638	16	1,693	46,243	74,960	35·18
Boksburg	24,380	1,149	36	1,396	36,185	63,146	38·61
Kimberley	20,480	1,075	707	12,991	26,452	61,705	33·19
Vereeniging	17,255	684	43	647	41,040	59,669	28·92
Potchefstroom	16,731	447	16	1,456	13,224	31,874	52·49
Uitenhage	14,246	356	648	6,684	16,299	38,233	37·26
Parow	13,270	95	38	6,285	616	20,304	65·36
Randfontein	12,675	52	2	797	21,072	34,598	36·63
Paarl	11,951	39	546	14,683	2,433	29,652	40·30
Vanderbijl Park	11,782	18	4	68	10,967	22,839	51·59
Goodwood	11,684	870	1,159	29,402	3,952	47,067	24·82
Kroonstad	9,977	—	2	818	14,546	25,343	39·37
Worcester	9,208	80	505	10,352	4,709	24,854	37·05
Klerksdorp	9,118	434	49	923	13,665	24,189	37·69
Bellville	9,065	53	184	5,626	2,080	17,008	53·30
Epping Garden Village	8,873	3	—	50	7	8,933	99·33
Alberton	8,675	88	—	241	6,465	15,469	56·08
Grahamstown	8,616	175	31	2,828	11,694	23,344	36·91
Queenstown	8,605	101	—	2,217	14,643	25,566	33·66
Pretoria North	8,500	83	—	15	1,069	9,667	87·93
Stellenbosch	8,271	77	318	7,122	1,934	17,722	46·67
Oudtshoorn	8,239	21	13	9,143	1,162	18,578	44·35
George	8,200	4	7	4,614	413	13,238	61·94
Walmer	8,168	219	80	6,204	4,780	19,451	41·99
Nigel	7,456	300	32	98	22,486	30,372	24·55
Pietersburg	7,272	697	10	216	11,640	19,835	36·66
Rustenburg	7,232	602	15	244	6,210	14,303	50·56
Bethlehem	6,990	5	2	437	11,082	18,516	37·75
Pinelands	6,512	—	9	497	347	7,365	88·42
Welkom	6,414	—	1	8	3,667	10,090	63·57
King William's Town	6,341	106	20	1,609	4,280	12,356	51·32
Strand	5,782	81	940	3,048	590	10,441	55·38
Edenvale	5,569	58	1	226	6,050	11,904	46·78
Ladysmith (N)	5,439	3,027	14	381	7,455	16,316	33·33
Witbank	5,228	169	1	127	10,369	15,894	32·90
Malvern	5,126	3,187	21	414	2,453	11,201	45·77
Graaff-Reinet	4,918	40	2	5,754	3,339	14,053	34·99

In Johannesburg, by designing on the original basis of 2·5 cubic feet per capita instead of 4 cubic feet per capita, the combined works would be approximately 1½ million cubic feet of digestion capacity short of that required to produce completely digested sludge and render non-viable all worm ova, cysts, etc. This is another case where pure research has proved to be of inestimable value to the designer of engineering projects.

Rainfall and Sewage Disposal Problem

The quantity and intensity of the rainfall has a great bearing on the design of sewage disposal works. The amount of run-off in any area determines, within certain limits, the amount of water draining to the rivers, and the flow of the rivers determines the quality and quantity of effluent which can be discharged into rivers.

In South Africa the amount of rainfall is variable : some areas have a rainfall of less than 5 inches per annum ; in more than 64 per cent of the area of the Union of South Africa the rainfall is less than 20 inches per annum and more than 46 per cent has a rainfall of less than 15 inches per annum. In one of the towns, where the Author acted as a consultant, the annual rainfall averages less than 1 inch and yet on one occasion 4·6 inches fell in less than one hour.

There are in South Africa distinct areas where rain falls mostly during the summer months and other areas where rain falls mostly during the winter months. The intensity of rainfall in the " summer rainfall areas " is very much greater than in " winter rainfall areas." Usually the water-table in winter rainfall areas is much higher than in summer rainfall areas and this materially affects the infiltration water into the sewer. Completely separate systems are used for sewage and storm-water. In 5 drainage areas in Johannesburg the ratio of maximum run-off of storm-water to the maximum flow to the sewage disposal works is 2,000 : 1.

In Mauritius the distribution of rainfall is worthy of recording here. The island is roughly 35 miles long or approximately equal to the distance between Reading and London. If Reading is assumed to be the southern tip of the Island and London the northern tip, the average annual rainfall would be 30 inches at Reading, 180 inches at Slough and 15 inches in London !

In South Africa the Author has known 5·25 inches of rain to fall in 25 minutes—an intensity of 12·6 inches per hour. Under cyclonic conditions in Mauritius a rainfall of more than 146 inches in 60 hours has been experienced !

Water-Table and the Sewage Disposal Problem

The amount of infiltration water flowing into the sewer depends upon the following factors :

(*a*) Water-table.
(*b*) Soil temperature.
(*c*) Sewage temperature.

There are many cases, where the water-table is high, where there is practically no filtration of water into the sewer because the soil temperature gradient is such that there is very little difference between the temperatures of the soil and sewage throughout the year. This is so in many parts of Mauritius. On the other hand, in winter rainfall areas in South Africa there is a big difference between soil temperature and sewage temperature. This difference results in hair cracks developing at the sewer joints and there is infiltration at an average of 1 cubic foot per minute per mile of sewer or 9,000 gallons per day per mile of sewer.

Temperature and the Sewage Disposal Problem

The following factors must be considered :—

(*a*) What is the range of atmospheric temperature ?
(*b*) What is the soil temperature gradient at, say, 10 to 12 feet below ground level. Is this temperature gradient constant ?
(*c*) What is the estimated difference in temperature between the soil and sewage ?
(*d*) What is the average temperature of the sewage arriving at the sewage disposal works ?
(*e*) What are the maximum and minimum differences between the sewage and the atmosphere ?

Water Supply and the Sewage Disposal Problem

From an examination of the records of 164 local authorities' water supply, 105 of them have a water supply which is only sufficient to supply 15 gallons per day, or less, per head of population. In addition to purely domestic water, between 7½ and 9 gallons of water is required per head of population for sewerage purposes. Before it is possible to install a sewerage and sewage disposal scheme in many towns in Southern Africa, an augmented supply of water is essential.

The greatest concentration of population is in Johannesburg, and the area within 36 miles of Johannesburg ; 48 per cent of the total European population live in this area.

The water supply is taken principally from the Vaal River, which necessitates pumping more than 100 million gallons of water per day against an effective head of approximately 1,800 feet. The water is supplied to local authorities at approximately 9*d*. per 1,000 gallons.

By means of the barrage below Vereeniging and the Vaal Dam above Vereeniging there is a reserve supply for 3 years. The Chief Engineer of the

Rand Water Board, who spent the earlier years of his life on the banks of the Vaal River, can remember when it was possible to cross the Vaal River without getting his shoes wet.

South Africa is justly proud of the activities conducted by the Rand Water Board—a civil engineering project of no mean magnitude.

Co-operation in the Design and Operations of Sewage Disposal Works

Co-operation between the interested departments is to be desired in the design of sewage disposal works. There is no doubt it pays dividends.

There are so many factors to be considered. Local conditions are of the utmost importance : the local authorities' engineer may not be a specialist so far as sewage disposal is concerned, but he can be of great assistance to the consultant. The knowledge of the Medical Officer of Health and his staff should be carefully considered as well as the knowledge of the sewage works manager.

When designing new works the Author endeavours to use as much local knowledge and experience as he can gather ; for instance, during the design of the works in Mauritius, the Chief Medical Officer of Health, the Bacteriologist, the Director of Public Works, and the Financial Secretary were consulted extensively. By following this procedure the permanent officials knew, in detail, the reason for every step taken. On the other hand the Author was able to draw on their experience so far as local conditions were concerned—an absolute necessity for successful design.

It should be realized that works, when put into operation, are left to the care of permanent officials. These officials would welcome the help of the designer to enable them to know the reasons why the particular design was adopted. The Author has always maintained that it would be in the interests of the local authority and the consultant, if the latter were retained for a period of 3 years until the works are operating smoothly and satisfactorily.

Human relations are necessary in the management of sewage disposal works, as they are in industry—a factor which is often overlooked.

The late Mr J. D. Watson once said " with encouragement and guidance a manager of a very badly designed sewage disposal works can produce a better result than a manager of the best designed works who felt that no one was interested in the works."

In the British Medical Journal dated September 11th, 1948, Sir George Schuster wrote :—

> " Good human relations in industry can only be surely founded on the treatment of each individual as a human being of infinite value whose welfare in the highest sense must be regarded as an end in itself. Industrial employment can provide the foundation for this welfare to the extent that the worker can find interest, free self-expression, and

happiness in his work, something more than a mere distasteful way of earning a living. To help workers to find this should, therefore, be seen as an essential part of the manager's task in handling human relations."

Research Work on Sewage Disposal in South Africa

Organized research on sewage disposal problems did not commence in South Africa until 1933. Some isolated work had been done before this by the late Mr D. E. Lloyd-Davies and Mr Croghan on grasses, brushwood filters, and Imhoff Tanks, whilst Mr Lundie, the Bio-Chemist at Pretoria did work on separate sludge digestion. The Author worked on the treatment of wine wastes.

When considering the new works for Johannesburg, the Government of South Africa asked the City Council to organize research work on the "activated sludge" process. The Council agreed and three small plants were installed, each capable of treating 100,000 gallons of sewage per day. The plants were designed on (*a*) the diffused air principle, (*b*) on the Simplex principle, and (*c*) the Spiroflow principle. These plants were fitted with flow recorders and were completely independent plants, although fed with the same settled sewage.

The plants were operated by the City Engineer's staff under the direction of Dr Harold Wilson, for a period of 2 years, and the best results were obtained from the diffused air plant. It is quite possible that if the experiments had been conducted at sea-level and at a place where there was comparatively low evaporation, instead of in Johannesburg at an altitude of between 5,000 and 6,000 feet above sea level, a different result would have been obtained.

At this time the City Council appointed a team of research workers, which included civil engineers, chemical engineers, chemists, and biologists, and the results of their work have been published in the journals of many scientific bodies. The most important results, so far as the subject under review is concerned, are :—

(*a*) The continuation of the work, commenced by the Author in 1928, on re-circulation.
(*b*) The ventilation of biological filters.
(*c*) The comparison of an open 6-foot percolating filter with an enclosed, artificially ventilated, Prüss filter 12 feet deep.
(*d*) The effect of temperature on purification in biological filters.
(*e*) The effect of depth of media and various grades and kinds of media.
(*f*) The effect of horizontal ventilation zones on the purification by biological filters.
(*g*) Investigation into the commercial use of sewage sludge gas.
(*h*) Slow sand filters for the final purification of sewage.

The research department of African Explosives was interested in (9) above and compressed sludge-gas was supplied by the City Council of Johannesburg to the company. Co-operation between the two bodies resulted in the production of cyanide from methane and ammonia. Cyanide is extensively used in the gold mining industry and was previously imported. A cyanide factory was established on one of the sewage disposal works, which is adjacent to one of the City Council's power stations; cheap electrical power was available so that gas was not required for the generation of electrical energy and the cyanide factory was able to supply hot water for circulation in the sludge digesters.

The City Council is now receiving about £12,000 a year from the sales of gas to the Klipspruit cyanide factory which provides about half the quantity required in the gold-mining industry, and this revenue is expected to reach eventually about £40,000 per annum. Further, all money voted for research of all kinds, in connexion with sewage problems, has been recovered.

This could not have happened had the City Council not acted on the recommendation of the City Engineer and the City Medical Officer to provide an extra chemist on the staff of the Laboratory Division, over and above the number needed to cope with the ordinary applied work of the division, to allow continuous research to be maintained.

Even in South Africa few investments have ever yielded so great a dividend, since most of the capital invested is being returned yearly as income with very little annual cost.

Co-operation with the South African Institute of Medical Research has yielded splendid results in the investigation of the effect of internal parasites, etc., on the sewage disposal problem; details of one of these effects—Poliomyelitis—is given above.

Considerable work has been done on slow sand filters as applied to sewage purification work at Johannesburg. The type developed by Mr C. H. Hamlin at Luton, has been installed with great satisfaction. Dr Wilson and Messrs Vosloo and Heynike have carried out extensive work on rapid gravity and pressure filters.

The following local authorities have research staffs and have contributed in no small manner to the solution of sewage disposal problems in South Africa. Cape Town, under Mr Croghan and his successor Mr Abbott; Pretoria, under Mr Lundie and his successor Mr de Waal; and Germiston, under Dr Murray and his successor Mr Krige.

The South African Council of Scientific and Industrial Research, through Dr Wilson and Dr Stander, have continued the work commenced by the Author on the anaerobic digestion of fermentation waste. So great is the aggregate possible yield of gas from the sugar and fermentation industries that, where not needed for fuel, this gas would form a basis for a large synthetic-chemical industry of great importance.

Reference has already been made to the work of Dr A. R. P. Walker on diet. Without his valuable contributions it is questionable whether the

solution of the sludge digestion problems would have been solved so quickly.

In closing this section on Research in South Africa, the Author wishes to stress, and draw to the attention of Government and Local Authorities, Members of Parliament and City or Town Councillors, as well as the lay public, a fact, which, though often recorded in respect of other activities, has not been publicly stated by workers in the field of sewage and industrial waste treatment.

This fact is: that fundamental research work, that is, research on a theoretical problem, with or without any immediate intention to apply the result, has time and time again proved of tremendous financial benefit to those bodies enlightened enough to provide personnel and facilities for fundamental research.

Future Sewage Disposal Schemes

Because of the intensity of the rainfall it is extremely doubtful whether a combined scheme will ever be installed in Southern Africa, but many millions of pounds will be spent on the control of stormwater.

Every local authority except one, with a population of more than 15,000, either has a complete sewerage and sewage disposal scheme or has one in course of construction. The only exception is a non-European township with a population of approximately 60,000, on the outskirts of Johannesburg. This area will be drained to the proposed Johannesburg northern works referred to later. Most of the future schemes will be for a maximum flow of 1 million gallons per day.

The City of Durban, Natal, contemplates, at an early date, spending approximately £500,000 to treat its sewage. This has become essential because of the adverse effect on the bathing beaches, since, at present, crude sewage is discharged into the sea.

The City of Port Elizabeth, Cape Province, must construct a new outfall sewer discharging at least one mile out to sea in order to obviate the present unsatisfactory conditions.

Because of the tremendous development of Johannesburg and the consequent increase in land values, the City Council has resolved to close down three of its present sewage disposal works and drain the sewage to one works situated about 18 miles from the city centre. The new works will take, in addition to the sewage from the three works to be closed down, the sewage from ten smaller local authorities. The cost of the new works plus the outfall sewers will be more than paid for by the sale of the land on which the present works are situated, originally bought by the City Council for one-fiftieth of its present value.

Future Treatment of Sewage in Southern Africa

As mentioned above, there is a shortage of water in South Africa, greater use must be made of effluents from sewage disposal works, and it is quite possible that it will be made a condition that all effluents must be sand filtered before being discharged into a watercourse. The quality of sewage effluents may have to be better than the standards laid down by the Royal Commission.

From the Author's experience it would appear to be desirable to construct a storage dam having a capacity of at least 7 days' average flow, and pass the effluent through the storage dam before discharge into a watercourse.

It is probable that, from an economic point of view, greater use will have to be made of sludge gas. On the contemplated works to serve the new goldfields in the Orange Free State, sludge gas will be used for the manufacture of cyanide, which is essential to the gold-mining industry.

In most of the larger centres, sewage effluents will be used as circulating water in power stations and industry, as is done at present in Johannesburg and Pretoria. This will effect considerable savings, for it is not generally realized that between 1·25 and 1·5 gallon of water are required to generate 1 unit of electricity in steam power stations.

The Use of Waste Products

Very great developments from the better use of waste products from industries may be expected in South Africa, as elsewhere.

Arising out of work done by the staff of the Johannesburg City Council, together with more recent work of the Water Treatment Unit of the S.A.C.S.I.R., it is now clear that better use may be made of the effluents from fermentation and food industries than to turn the wastes into sewers, into streams, on to land, or into the sea.

The considerable quantities of gas recoverable by controlled anaerobic digestion of these wastes should suffice to evaporate the residual liquid for recovery of potash, if this is necessary, but the gas would alternatively provide methane for large developments of chemical synthesis and dry ice for varied use.

The limits of application of anaerobic digestion cannot even be guessed at now. Applied to cellulosic wastes of all kinds the possibilities are tremendous and world-wide.

The sugar industry produces a vast quantity of bagasse, the crushed-up fibre of the sugar cane, which in the process of sugar recovery is well cleaned. In South Africa and elsewhere this bagasse is largely used as boiler fuel, though a little is used for making fibre boards. In view of the world shortage of paper it would seem urgent that this fibre should be used for paper making. This is no mere suggestion, for already good-class newsprint is being made in America from sugar cane bagasse.

Details of Works

The Author has purposely limited the number of plans in the Paper presented, to three :—

Figs 1, Plate 1, give details of a screening chamber and detritus tanks.
Figs 2, Plate 1, give details of a dividing tank.
Figs 3, Plate 2, give details of the " Charles Hamlin " sand filter.

As has been previously mentioned mechanization of sewage disposal works, because of the labour position, has been kept to the minimum.

There would appear to be no point in giving details of sedimentation tanks, percolating filters, humus tanks, etc., but the following details may be of interest.

Screens

There are but a few towns where mechanical screens are installed, the principal reason for this being the difficulty of getting Africans to undertake the work from a religious point of view. South African experience shows that mechanical screening apparatus is more expensive to instal and operate than bar screens.

When installing bar screens, particular attention must be paid to the setting angle of the screen, which should never be more than 30 degrees. If this angle is exceeded there is a great danger of the sewage backing up the sewer and becoming septic. Neither the Bantu (African) nor the Indian uses the soluble material used generally by the European. If the correct angle of the screen is used the flow of sewage will itself keep the screen clean.

Detritus Channels

In order to obviate the necessity for dredging, or otherwise removing detritus mechanically, the Author developed the type of detritus chamber shown in Figs 1, Plate 1.

The channels are designed to give a constant velocity. An extra channel is constructed, and it will be noticed that each channel is underdrained.

The method of operation is to shut off a channel at a definite time each day, open the drainage valve, and allow the channel to drain until the next morning. The man who looks after the screens then removes the comparatively dry detritus for disposal. The cleaning takes less than an hour.

This method of detritus removal has been successfully operated for nearly 20 years on a sewage disposal works treating 20 million gallons a day, and other works dealing with between 1 and 5 million gallons a day use the same method.

Dividing Tank

Details of the dividing tank are shown in Figs 2, Plate 1, which are self explanatory.

It is used whenever the flow has to be divided into four or more equal parts. On one works it was used to divide the flow :—

(*a*) Equally between sixteen sedimentation tanks with a capacity of 1¼ million gallons per day each.

(*b*) Equally between sixteen humus tanks with a capacity of 1¼ million gallons per day each.

The dividing tank was designed to have a retention period of 15 minutes. When first put into operation it gave a great deal of trouble because of the amount of sediment deposited, but at that time it was operated as an upward-flow inlet. Then the inlet was altered to the way shown in *Figs 2* and from the date of the alteration until to-day—approximately 15 years—it has never given a moment's trouble.

The " Charles Hamlin " Sand Filter

General

The " Hamlin " filter is a type of slow sand filter which has been used with considerable success in Johannesburg.

It differs primarily from the type of slow sand filter designed by the late Sir Alexander Binnie for Salisbury, Wiltshire, and introduced by the Author on sewage works in South Africa, because it is possible to reverse the direction of flow through the filter to facilitate the cleaning of the filtering medium, thereby enabling this cleaning to be carried out more expeditiously and efficiently. The wastage of surface sand—which is often quite considerable during cleaning of the usual type of filter—is reduced to the minimum.

The Author's late uncle operated, for more than 30 years, the original sand filter in Salisbury, and this later improved type was developed by the Author's cousin—late Sewage Works Manager at Luton—and named, by the Author, after him.

Description and method of operation

Figs 3, Plate 2, gives details of the type of filter in use at Klipspruit Disposal Works. As will be seen, the filtering medium is a layer of fairly coarse sand 6 inches deep. Below this are three layers of stones of increasing size which allow easy flow of the liquid to the two collecting channels at the centre.

The normal method of operating the filter is as follows :—

The effluent from the humus tanks is admitted from the supply channel via the 15-inch-by-15-inch penstock (D) into the top centre distributing channel, which runs the length of the bed. From the end nearer the supply channel a 12-inch-diameter outlet pipe controlled by a valve (H) is provided. Normally this valve is closed and is only used when the filter is washed.

From the distributing channel, the effluent spills over on to the surface of the sand. It then passes through the sand and the stone below it to the

two central collecting channels and out through the 6-inch-diameter discharge pipe (E).

This bed will handle efficiently approximately 250,000 gallons of liquid per day.

At the beginning very little head is required to overcome the resistance offered to the passage of the liquid through the sand, but as the blanket of humus develops on the surface and the interstices become clogged, the depth of water above the sand increases if a constant rate of flow is maintained. When the depth of water is about 15 inches it becomes necessary to clean the filter. This operation is usually carried out every 24 hours at Klipspruit Sewage Works.

Cleaning operation

When it becomes necessary to clean the filter, water from an adjoining filter is admitted into the channel on the outlet side of the filter. The level in this channel is maintained 3 inches above the sand in the filter. Penstock (D) is then closed and penstock (F) and valve (H) on the 12-inch sludge line are opened.

The flow then passes in the reverse direction through the lower central channels and up into the upper central distributing channel. From this channel the liquid flows into the 12-inch sludge pipe and on to the sludge digestion tanks. At the same time a travelling screed is moved up and down the filter along the sand surface at a brisk pace to create a wave in front of the screed. This agitates the surface and assists in transmitting the humus to the central collecting channel. This operation is continued for between 10 and 15 minutes, by which time the liquid is normally flowing freely through the bed and no more humus is coming off.

Penstock (F) and sludge valve (H) are then closed and penstock (D) is opened, and the filter is again ready for operation.

It is normal practice at Klipspruit Sewage Works—where there are thirty-two of these filters—to operate in batteries of eight, each bed being operated for 3 weeks and then rested for a week. This method has been found, after experiment, to give the best results ; it prevents the formation of algae in the filter media.

When the bed is cleaned for the last time before resting, it is drained of all water by opening the 6-inch valve (G) which also discharges into the sludge line. The bed is then carefully raked and smoothed over, and any sand losses made good. Any sand which has been washed over into the sludge line during cleaning is intercepted and recovered in a sand trap which has been installed in the sludge line. Sand losses are thus almost negligible.

Conclusion

The Author has not attempted to give details of various disposal works in South Africa, because to discuss them in detail would make the Paper

unwieldy. As stated above " there is no best method of sewage disposal which will meet all the various conditions pertaining in various communities."

It is realized that much more could have been written on the subject, but he hopes that sufficient information has been given to indicate some of the problems to be found in sub-tropical countries. These problems can only be solved by the co-operation of Medical Officers of Health, Engineers, Chemists, Biologists, and last, but certainly not least, Sewage Works Managers.

It is hoped that some of the larger and richer local authorities will provide the funds to enable trained research workers to investigate special problems. They owe a debt to the pioneer in this branch of Public Health Engineering, for their sewage disposal works have been designed on the results obtained by these pioneers.

The Paper is accompanied by three sheets of drawings from which the folding Plates 1 and 2 have been prepared.

Discussion

The Author introduced the Paper with the aid of a series of lantern-slides.

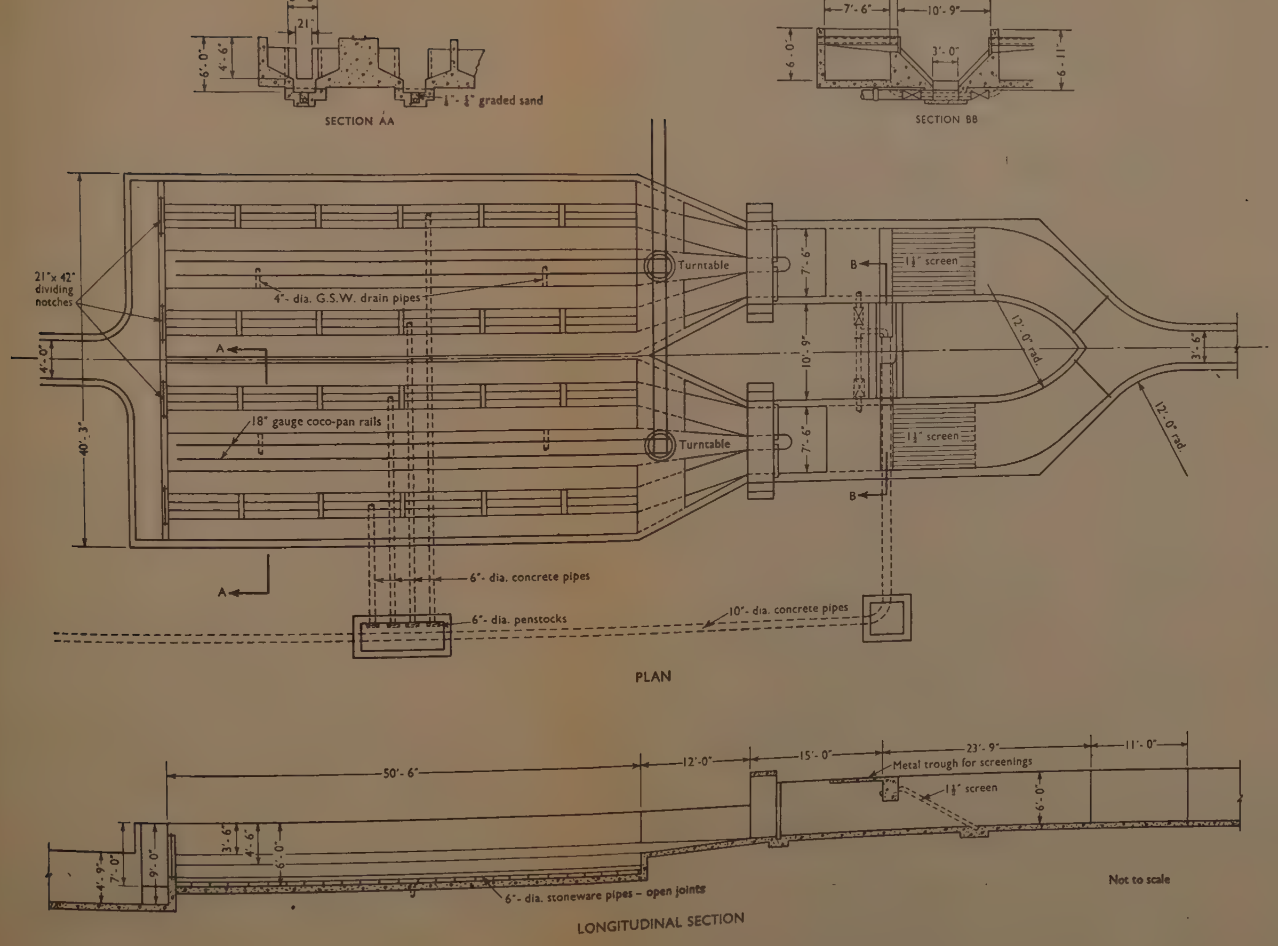
Mr F. C. Vokes said that 25 years ago he had had the privilege of working with one of the Author's pupils at the works of the Birmingham Drainage Board. Anyone who had had the same privilege was singularly fortunate. Since that time he had been greatly interested in the work the Author had done as City Engineer of Johannesburg and afterwards in a still wider sphere.

The Author had expressed the view that the information in the Paper might not be of any direct assistance to public health engineers practising in Great Britain. Mr Vokes assured the Author that they could not fail to derive benefit from a careful study of the Paper as they tackled their present difficulties.

It had been stated in the Paper that, in South Africa, sand filters had been of inestimable value in reducing the incidence of disease, but that in only three cases had permission been granted for effluent to be discharged, after passing through sand filters, direct into the stream. The Author had also stated that from his experience it would appear to be desirable to construct storage capacity for at least 7 days' average flow and to pass the effluent through that before discharge into a watercourse. Mr Vokes assumed that the storage was not provided for the purpose of evening out the flow in the river. Was it provided as a safeguard in case of a breakdown of the sand filters ? Was it for the purpose of reducing the bacterial count, or for reducing the biological oxygen demand of the effluent ?

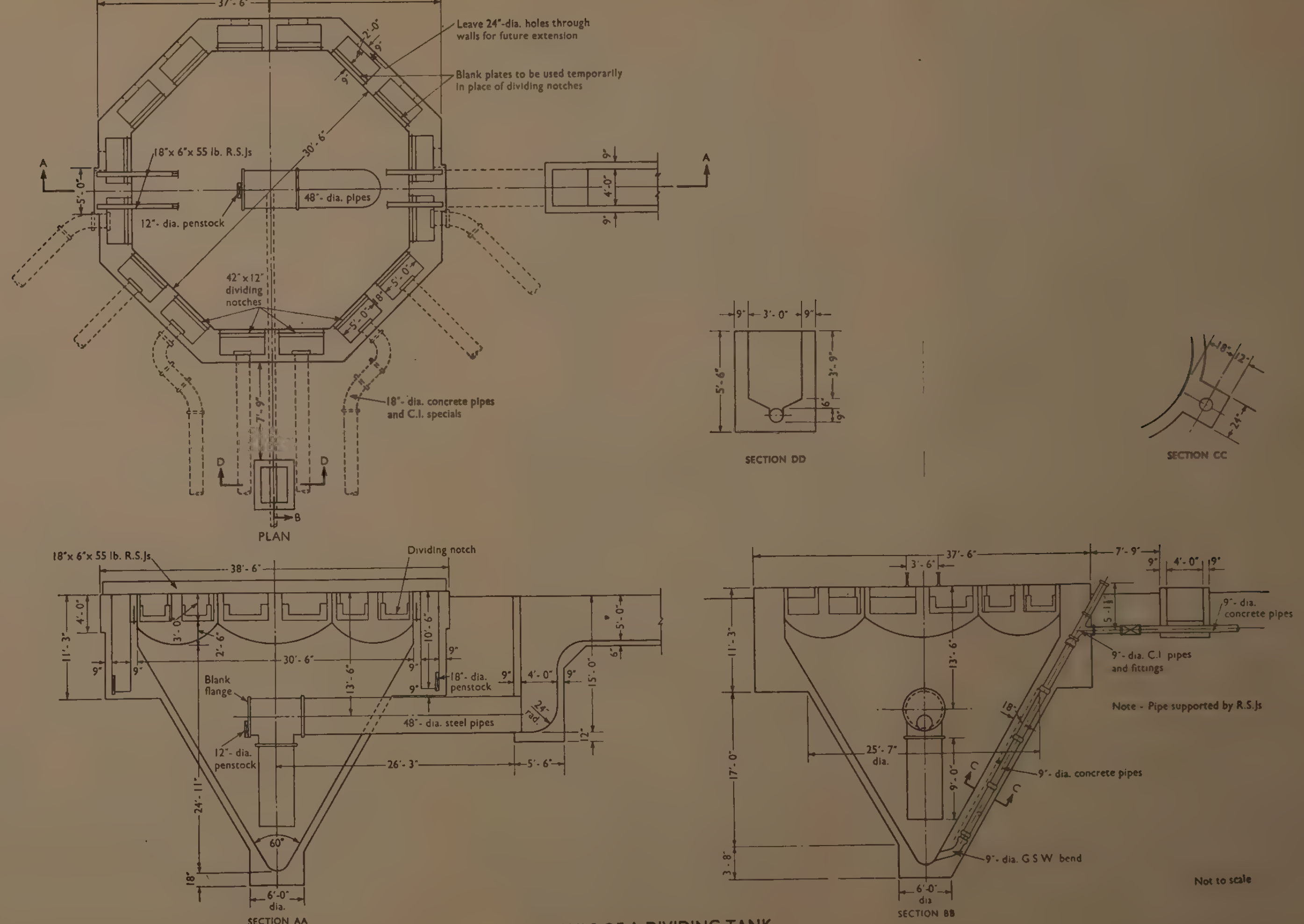
SEWERAGE AND SEWAGE DISPOSAL IN SUB-TROPICAL COUNTRIES, WITH SPECIAL REFERENCE TO SOUTH AFRICA AND MAURITIUS

FIGS 1



DETAILS OF A SCREENING CHAMBER AND DETRITUS CHANNELS

FIGS 2

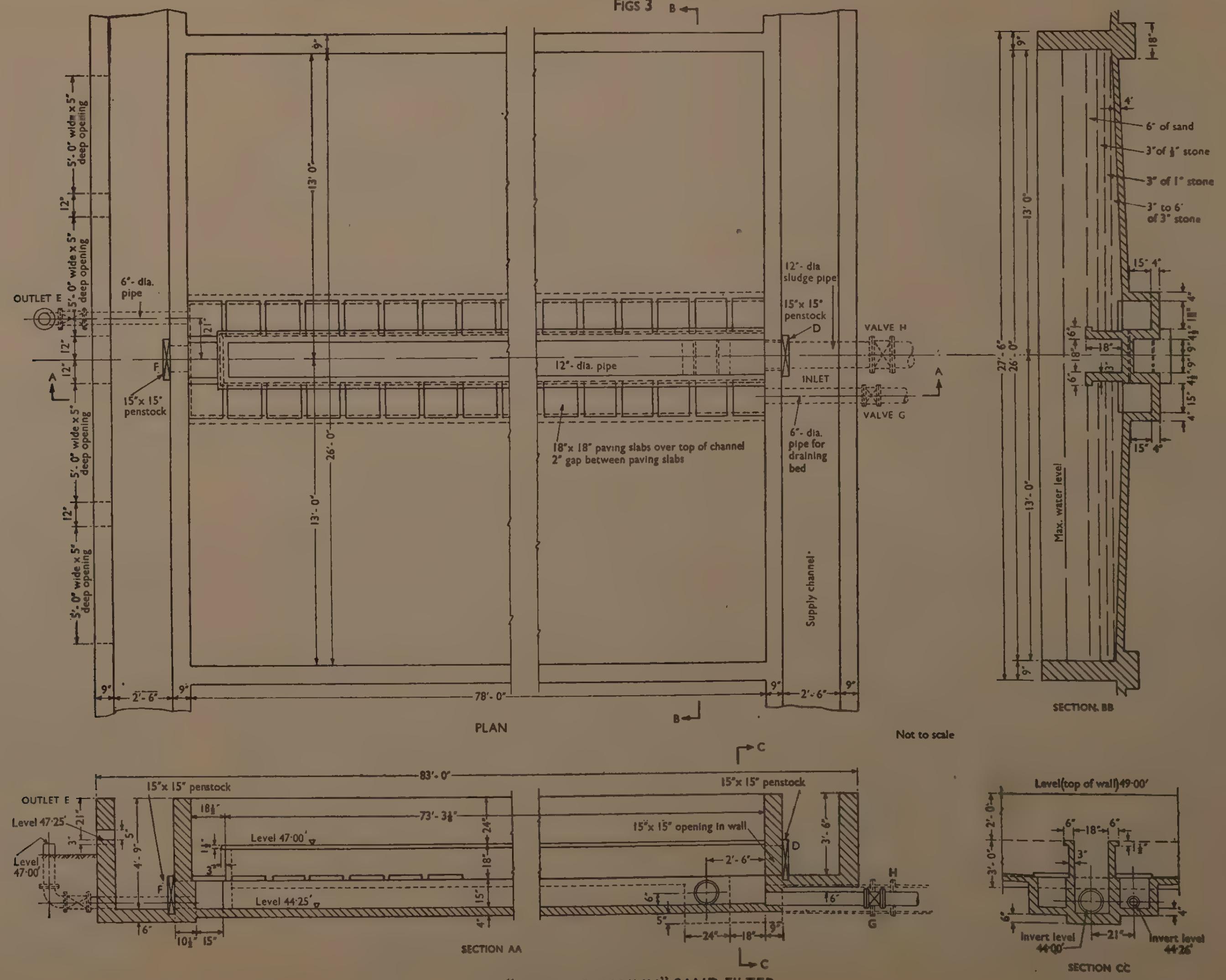


DETAILS OF A DIVIDING TANK

E. J. HAMLIN

SEWERAGE AND SEWAGE DISPOSAL IN SUB-TROPICAL COUNTRIES, WITH SPECIAL REFERENCE TO SOUTH AFRICA AND MAURITIUS

FIGS 3



THE "CHARLES HAMLIN" SAND FILTER

In 1910, the late Mr John D. Watson, afterwards President of the Institution, when referring [1] to the treatment of tank effluent on land, had said: "The whole farm of 2,800 acres may be regarded as capable of treating, year-in year-out, no more than 12,000,000 gallons per day without injury. Notwithstanding this, the farm actually treated more than 20,000,000 gallons per day during the years 1902 to 1906 inclusive, but the effluents steadily deteriorated under the strain."

The Author had stated that the Union Public Health Department had formerly forced local authorities to purchase 100 acres of land for each million gallons of sewage to be treated per day, but that it now encouraged local authorities to buy four times as much land as before. Had the results formerly obtained with land treatment of effluent been so unsatisfactory as to necessitate that very large increase? If so, in what particular way had the land treatment failed?

Mr W. E. Blizard said that those who had lived in tropical countries would probably appreciate more than others the difficulties which were mentioned in the Paper. In the tropical countries in which he had himself been, there had generally been no sanitation at all as it was understood in Great Britain; there had been what he might call jungle sanitation, which might be effective in some ways but which was not altogether desirable.

There was a reference in the Paper to the reservations, and it would be interesting to know what, if anything, was being done in such reservations to improve their insanitary conditions.

He was interested to see on p. 564 a reference to the works at Pietermaritzburg becoming derelict. Apparently it must have been a wonderful works, and he could not understand why it should have become derelict. He knew that in Great Britain sometimes it was rather heartbreaking to go and look at works some years after one had constructed them!

On p. 565 there was a statement concerning the water table which it was difficult to grasp. There was reference to the water table rising considerably and to a legal fight, after which it had been decided to dig a 6-foot-deep trench. He could not see how that could have had the slightest effect, because if the ground was porous so as to let the water through to the subsoil, how could a trench improve that?

On p. 567 there was a reference to the 400 acres which local authorities had been forced to buy. Were they cropped in some way, or farmed; and if so, with what kind of crops?

With regard to sea outfalls, had aerial photography been used to follow the course of the sewage discharge into the sea? That had been done in Great Britain with very good results.

On p. 569 there was a reference to a filter 12 feet deep. Would it be possible to have a comparative analysis to show what the result was on

[1] "Birmingham Sewage-Disposal Works," Min. Proc. Instn Civ. Engrs, vol. clxxxi (1909–10, Part III), p. 259. See p. 264.

the effluent for a 12-foot-deep filter as compared with one of 6 feet depth or normal depth, such as in England ?

With regard to the storage of 7 days, it would be interesting to see analyses of the effluent before going into that storage and of the results after 7 days.

Last but not least, there was the question of the fly pest. Had any special precautions been taken to keep down the flies ?

Mr C. B. Townend said that the Author, in spite of his very busy life as City Engineer of Johannesburg, had always maintained a very close association with his colleagues in Great Britain and had not spared himself in the least in the service of the Institution, and of other bodies with which he had been connected.

The Author had succeeded in conveying the impression of a keen and vigorous young country very much alive to the importance of public health engineering, which in many respects was a much greater problem in sub-tropical areas than it was in Great Britain. During the evolution of sewage purification over the past 50 years, it was really remarkable how many instances could be found in South Africa of the very early application of developments and techniques in the subject. In certain respects South Africa appeared to be ahead of Great Britain, notably in the cleaning up of bathing beaches around the sea coasts ; and certainly the production of cyanide from methane for the gold-mining industry was in advance of any corresponding British process.

Conditions which allowed the large regional project of Johannesburg to be entirely paid for by the sale of the land on which the three displaced works were at present situated would arouse envy elsewhere. Much might have been done in Britain on similar lines, but it was now too late. For example, when the West Middlesex undertaking had been carried out, an area of about 1,000 acres had been left in the hands of local authorities which owned the twenty-eight redundant works that had been displaced. Some of the land at the beginning had been sold for as much as £2,500 an acre, and the total area collectively might have realised £2 million or so towards the £5½ million total cost of the scheme. However, the post-war legislation had altered all that, and the values of land released in that way had dropped very much on account of the very high development charges which were levied on any new use to which the land could be put, with the result that many of those sites were now liabilities rather than assets. In that respect, there was one advantage in being in a new country which had not quite reached the same degree of social development.

The main theme of the Paper was the necessity in sewage disposal for selecting the proper methods to suit local circumstances and how very much the local conditions in South Africa differed from those in Great Britain. The most striking difference, of course, arose from the shortage of water and its effect on the whole outlook regarding the disposal of sewage effluent. In Britain one often suffered from too much water, and the

problem of river pollution, which had existed for the past 100 years at least, was by no means cleared up even yet. In South Africa, apparently, river pollution had never started, because normally sewage effluents were not allowed to be discharged into the rivers, and even today there appeared to be only three cases of that method of disposal. Such control had been possible because of the wide open spaces of thirsty land which were available for purification.

In many respects that had been a most fortunate circumstance for South Africa, because she could now afford to sit down in a leisurely way and calmly determine what were to be the model standards to be imposed before discharge of a sewage effluent into a river should be allowed. The standards listed in the Paper appeared to be very comprehensive and to cover a far wider range of test than were normally applied in Britain. Standard No. 4 called for a limitation of dissolved solids which must be within such a figure that subsequent usage of the water should not be prejudiced. It would be interesting to know exactly what action an authority could take to reduce the dissolved solids if they were found to be too high.

Regarding the difference in the values of labour in the two countries, the Author had shown that it did not pay to mechanize many of the operations which were carried out in the South African works. Whilst that might be true under present conditions, it would be interesting to know whether the Author regarded it as a passing phase, or whether he considered it would always be so. Fundamentally, the standard of living of the backward races arose from their primitive methods of working, and if those standards were to be raised, mechanization would some day be necessary. On the other hand, the Author had described the outlook of the native, who apparently had no desire to raise his standard above a certain point. Yet again, current news from South Africa frequently mentioned the shortage of native workers and said that it might become a very serious factor in the economy of that country. Was not mechanization required equally from that angle?

Mr H. D. Manning considered the Paper to be of quite unusual importance and interest for two reasons. The first was the very strong emphasis which it laid on the genuinely public-health aspects of sewage disposal. The second was that the Paper dealt with conditions in South Africa and in sub-tropical countries. Hundreds of millions of people lived in sub-tropical countries, and, considering the increasing importance of those countries and the amount of work which was waiting to be done in countries where the conditions, if not the same, were at least similar in many respects, it was obvious that the Paper was very timely and of great value.

Could the Author quote any statistics indicating the improvements in health in South Africa over the last 20 or 30 years. Mr Manning recalled the Presidential Address of the late Sir Roger Hetherington,[1]

[1] J. Instn Civ. Engrs, vol. 29, p. 1 (Nov. 1947).

in which Sir Roger had shown what the public health engineer had done over a long period in the way of improving the health statistics. Mr Manning's own search at South Africa House had failed to find similar statistics; he had gathered that there was difficulty in regard to non-European statistics, the recording of which had not been compulsory even in the towns until recently and even now was enforced only in urban areas.

There was one point which he hesitated to make, lest it should be misunderstood; but it seemed to him that in sub-tropical countries (he was not speaking of South Africa, which he had not had the pleasure of visiting) there were cases where very elaborate water supplies and sanitation had been developed for European settlements—quite rightly and very naturally—and important defence measures taken against the peculiar conditions of those countries, whereas the development of similar measures for the non-European communities seemed to lag behind very considerably. That had always seemed to him to be a little strange. One might visit a magnificent bungalow, where everything was laid on, with hundreds of gallons of water being used each day, and with excellent sewage disposal, but that same bungalow would be visited by native cooks and domestic workers, who in many cases, were still living in very unsatisfactory sanitary conditions. The relevance of that matter to the Paper was to be found in the Author's reference, on pp. 572 and 573, to the migratory native population which moved to and fro between the towns and the native reservations.

Mr Manning wondered if the time had yet come for the emphasis to swing away from elaboration and improvement of the works for large cities in the newer countries to dealing on simple lines with sewage disposal in the much more difficult and scattered native communities.

Mr C. D. C. Braine said that he had been associated with South Africa in one way or another for about 20 years and he had actually had the pleasure and privilege of working under the Author in that country.

The Author had indicated the debt which South Africa owed to Messrs Jameson and Lloyd Davies in respect of sewerage and sewage disposal, but it was Mr Braine's own belief that engineers in South Africa would wholeheartedly agree that South Africa owed far more to the Author today than to either of the two veterans whom the Author had mentioned.

The Author was unique, perhaps, in that, as City Engineer at Johannesburg, he had persuaded the Council to undertake a considerable programme of research in connexion with sewage disposal and, as was well known, both the City and South Africa had been amply rewarded by the results of that work which he had initiated and which had gone on ever since. In that matter, the Author practised what he had preached in the Paper, namely, the acceptance of "calculated risks," and his policy had paid high dividends. The same thing would also pay in Great Britain.

Mr Braine agreed absolutely with the Author's views on the urgent necessity in a dry country of using purified effluent for irrigation purposes.

The design of the Tel Aviv and Region Sewerage and Sewage Disposal Scheme would shortly be complete, and that scheme provided for the disposal on land of a dry-weather flow of 60 million gallons per day of purified effluent. That effluent was actually to be used for irrigating the loess lands in the Negev about 60 miles away.

The value of well-purified effluent was twofold. There was its value as perennial water in dry countries, and in addition there was its value as a fertilizing agent. In round figures, every million gallons of good effluent would carry about 3 cwt of fertilizer in solution and 2 cwt of humus in suspension. That small but constant load spread evenly day by day over plots of land had a surprisingly high monetary value.

A good effluent also presupposed the existence of dried sludge, which was of the utmost value in countries where the soil was starved of humus. Years ago at Johannesburg there had been keen competition between market gardeners for effluent from an activated sludge plant, but although that was indicative of the value of the effluent, it was not scientific evidence! Could the Author give actual comparative figures to illustrate the extra yield obtained from crops irrigated with purified effluent instead of with ordinary water? In the High Veld in South Africa, many streams ran fairly freely at the end of the summer rains, but during the winter months, which were bitterly cold and very windy, evaporation rates were very high and the land became parched and the spruits quickly dried up. The supply of sewage effluent, however, from a large town remained more or less constant all the year round and was a godsend to local farmers.

Climate affected sewage disposal in a number of ways. In hot weather sewage tended to become septic and concrete sewers suffered accordingly. The sewers in Cape Town were a case in point. Experiments in the United States had shown that eddies caused by velocities of only 3 to 4 feet per second in sewers sucked enough oxygen into the sewage to delay septicity for comparatively long periods, thus saving the fabric of the sewers from destruction, and it would be interesting if the Author could record evidence of the same effect in South Africa.

Droughts and water shortages were extremely common in that country and it would also be interesting to learn if any sewerage troubles had ever arisen owing to a lack of water in the sewers themselves; if so, what was the dry-weather flow per head of population at that time? Mr Braine had been much exercised by that problem some years ago, when water rationing had been proposed in several large cities in Great Britain. The proposed rationing would have been so stringent that it had seemed to him that there was a very real danger of the sewers in some cities ceasing to function owing to lack of water. In Britain the dry-weather flow at sewage works was usually a good deal in excess of the local water-supply, but in South Africa and in the sub-tropics generally the reverse occurred because much of the water was used for gardening, particularly in the dry reason. In Britain the diurnal variations in flow were fairly constant all the year round,

but that was not always the case abroad. There, in cold weather, the diurnal variation might be much the same, but in the hot weather, instead of a single peak flow occurring about mid-day, there might be two peaks: one about breakfast time and a second in the evening, the average daily flow in summer being about 50 per cent higher than the corresponding flow in winter. Since the climatic conditions at Cape Town, Kimberley, Johannesburg, and Pretoria were so very different, the Author's comments on a comparison of diurnal and seasonal variations in flows in those places would be instructive.

As the Author had already pointed out, the effect of high-intensity storms was such that almost all sewerage systems in the sub-tropics and tropics were rigidly separate. Nevertheless, the effect of rain on even the rigidly separate sewer might be very marked and there, as in Britain, the older the system the greater was the effect. Some years had elapsed since the Author had built many miles of local and main sewers in Johannesburg, and Mr Braine's own recollection (for he had been on that work) was that they had all been designed to carry four times the dry-weather flow. Infiltration had been reckoned to be negligible. Could the Author say what happened nowadays in wet weather and how many times the dry-weather flow was actually discharged in times of heavy storm?

Mr Joseph Rawlinson said that, in view of the Author's statement that the sewers in Johannesburg were only 7 inches in diameter (that had been some years ago, of course), he felt eternally grateful to one of his predecessors, Sir Joseph Bazalgette, C.B., Past President I.C.E., for constructing the mighty intercepting sewers across London 100 years ago. Would the Author say just when the reconstruction had taken place and what the size of Johannesburg had been before the sewerage system had been brought up to date? It would also be interesting to have some figures.

There had been a reference to the storm-water system. Could the Author say what the run-off was and what was the minimum time of concentration? Mr Rawlinson was quite sure that it differed considerably from London, but the figures would be interesting.

The Author had said that the production of gas brought in an income of about £40,000 a year for Johannesburg. That was a very high figure. The London County Council was now embarking upon a scheme of sludge digestion and the production of methane gas for providing all the power at the new works at Beckton. The production was likely to be about 4 million cubic feet of gas per day. Could the Author say just what expenditure of money was incurred year by year in order to produce an income of £40,000? What was the calorific value of the gas?

There was a reference in the Paper to the need for closer co-operation in research work. Mr Rawlinson would remind the Author, whom he thought probably knew it already, that at Beckton experiments were being made with the sludge-freezing process which so reduced the time for

drying that it could now (experimentally) be carried out a hundred times more quickly. If the Author had not seen the experimental plant, he would be very happy to take him over it before he returned to South Africa.

Lastly, could the Author give some information on what control local authorities had in South Africa over the discharge of trade wastes into the public sewers?

The Author, in reply, expressed his thanks for the manner in which his Paper had been received.

Mr Vokes and Mr Blizard had asked about the 7-day storage. About 27 years previously the Author had carried out some research work on that question. He had wondered if the results obtained by the late Sir Alexander Houston on raw river water would be repeated so far as sewage was concerned. He had been pleasantly surprised with the results obtained—the B.O.D. had been reduced and the bacterial count considerably reduced, except where *Bacillus Coli* was concerned. Upon investigation it had been proved to be due to bird and fish life. If a storage dam was to be incorporated in the design one thing was essential, namely, a humus-free effluent; the best way to get that was with the introduction of sand filters.

There was no doubt that sand filters reduced the bacterial count considerably. Dr Gear's researches—as summarized in the Paper—had proved that.

The Author looked upon sand filters as controlled land filtration and although (as stated in the Paper) the Department of Public Health had given permission to only three local authorities to discharge into a stream, the effluent which had passed through slow-sand filters produced an effluent which could not "pollute any natural stream, pond, or lake," and was discharged quite safely. The Author wished to stress that the effluent had to be humus-free, or sludge banks would be created and would cause a nuisance.

The Union Public Health Department had at one time insisted only on 100 acres per million gallons—a figure obtained from overseas practice. The figure meant 167 inches of water per annum. That had resulted in the production of a good effluent, but the Health Department did not want any effluent to reach the river and so had demanded 400 acres per million gallons.

The Author maintained that that request was unreasonable; it was almost impossible to obtain at an economical figure. Moreover, in South Africa water was too valuable to waste.

In reply to Mr Blizard, there was not only co-operation among South Africans, but those in South Africa were deeply grateful for the help which they had received from their colleagues in Great Britain. The Author thought it could be said that, even before Mr O'Shaughnessy had put in sludge digestion at Bath, Mr Walton Jameson—through the influence of Mr O'Shaughnessy at Birmingham—had had the first separate digestion

plant. The co-operation did not end with Birmingham, and although the Author would like to pay tribute to the help and inspiration which had been received from many people, he wished to pay a particular tribute to Mr Townend and his staff; it was necessary only to put a problem to them to receive all the help within their power to give.

It was the jungle conditions which were the cause of the trouble. In some tropical and sub-tropical countries, the people went to the bush and high evaporation and high winds spread disease. On Friday last, when the Author had arrived from South Africa, he had received a long letter saying that thirty-nine cases of poliomyelitis had been notified in one day in Mauritius; probably those would not have occurred if the works had been in operation.

South Africa was a relatively young country so far as sanitation was concerned. Further, as indicated in the Paper, many towns had insufficient water to enable them to have modern sewage disposal works. However, the National Housing and Planning Commission which financed housing made it a condition, wherever possible, that modern sanitation should be installed on all new housing estates for which it lent money—whether those estates were to house European, coloured, or Bantu population. In the Reserves very little could be done.

With regard to derelict works, he sometimes thought that engineers did not impress upon their clients the importance of sewage-disposal works and the importance of maintaining them properly. When the Author had been appointed to Johannesburg there had been only one works. It was designed to treat 1 million gallons per day but was actually endeavouring to treat eight times as much. Many town engineers rarely took any interest in the works under their control. He had, himself, been on works in the West of England and had asked the works manager how often the surveyor came down, and he had received the reply, " We have had five new ones, and I have not seen one of them." One could not expect the works to be up-to-date and well maintained if nobody took any interest in them.

On the question of the case in the Appeal Court and the 6-foot trench, the reply was that under the Public Health Act the Minister had the right to lay down the quality of effluents that could be discharged into a stream, but he had never exercised that right. In that case, lower down the valley there had been a rock barrier across and so the effluent discharged had eventually been stopped by the barrier and had built up a high water table. By cutting a 6-foot trench around the works and draining the top layer from the farm into the steam, a great deal of the water had been prevented from going down into the valley; and the high evaporation of Johannesburg (it was 72 inches on an average, and in some places the Author was designing works where evaporation was 140 inches a year) had lowered the water table, and in that way the trouble had been overcome.

Aerial photography had not been used to trace the sewage field, but there had not been an up-to-date map of Mauritius, and it had been

possible to persuade the Home authorities to have an aerial survey made and to have an up-to-date map. He had been surprised, on seeing the photographs of Port Louis, to observe the extent of the sewage field in that harbour. He had been out in a cutter and had measured the sewage field at Durban, and it covered several square miles.

Full details of the research work on 12-foot filters were given in the Proceedings of the Institution of Sewage Purification. They were too voluminous to reproduce here, but a 12-foot filter in South Africa could successfully treat $2\frac{1}{4}$ times as much per cubic yard as a 6-foot filter. Of course, site conditions had to be studied carefully to get the most economical results.

The problem of flies was overcome in South Africa mostly by re-circulation of the effluent.

In reply to Mr Townend, the Author would say that they were re-designing their works, not only because it paid them to go out midway between Krugersdorp and Pretoria, but because it paid from the national point of view not to have to import material for the production of cyanide if it could be done by sewage gas. It had not paid originally to go out 18 miles, because it had been possible to obtain land cheaply round the works.

The most important factor influencing the decision to establish new works was the menace to health to the people of Johannesburg by the small local authorities surrounding it. There had been established in the new drainage area a local authority for natives only on the boundary of Johannesburg. That area housed 80,000 natives and they had no sanitation. In addition, there were a number of settlements and several local authorities within the new drainage area.

The City Council could drain all those areas into the new works and could treat the sewage from those areas at a much lower figure than the separate local authorities could. By the sale of the land on which the present works were established the cost of a more up-to-date one could be met. It was possible to quote the success of West Middlesex.

The present Irrigation Act laid down three types of water—primary, secondary, and tertiary. The primary water was for human consumption and for cattle, and nobody was allowed to take any water out of the stream until all the primary users were satisfied. Nobody could use water for industrial purposes until all the secondary users (which comprised irrigation works) had been satisfied. Because of the attitude of the public health authorities and the restrictions imposed by the Irrigation Act, the Government decided that it should nationalize all water-supplies and in that way control the effluent from the sewage disposal works. One of the conditions to the establishment of the new works at Johannesburg was that it should supply 10 million gallons per day to an irrigation dam owned by the Government.

With regard to mechanization, to which Mr Townend had referred, the

position was peculiar in a mixed country such as South Africa. He did not think that for a great number of years the country would be short of labour. Some farmers were short of labour because they expected their people to work for far too little money. Many of those present would remember the time when farm labourers in England got 10 shillings a week and a cottage, but they got ten times as much now; yet there was still a shortage. Labour did not like the countryside in South Africa which was far more lonely than it was in England.

Mr Manning had referred to the strong emphasis which the Paper laid on public health. They were charged as engineers to carry out that dictum of better water-supplies and drainage. When the Author had seen in Mauritius thousands of cripples suffering from poliomyelitis on account of the insanitary conditions, and children with tummies full of round worms because of no sanitation, he had often remembered what he had learned at his mother's knee—"it was not the will of God that any one of these children should suffer." He made no apology for having laid emphasis on the public health aspect.

Although the Author had stressed the public health aspect he had not forgotten the economical aspect. Statisticians usually estimated the value of an average human life in sub-tropical countries at £150. On an average, each death represented 2 years' loss of work. That was to say, for every death there was a loss of work throughout the community by illness of 2 years for one man, say, 600 working days at 3 shillings per day, or £90.

The population of the area to be sewered in Mauritius was 150,000. If sanitation reduced the death rate by only five per thousand—which was only one-third of the improvement recorded in Port Louis by the installation of sewerage, there would be a saving of 750 lives per annum.

750 lives at £150	£112,500
Corresponding loss of labour (750 at £90) . . .	£67,500
	£180,000

Capitalized at 5 per cent Interest and 1½ per cent Sinking Fund (loan period: 30 years), that figure represented the capital charges on approximately 2¾ million pounds, which was 50 per cent greater than the total cost of the Scheme.

It was not possible for the Author to give absolutely correct vital statistics for the total population of South Africa. It had only just been made compulsory for the Bantu to register births and deaths. Speaking generally, however, a conservative figure of the improvement resulting from the installation of sewerage and sewage disposal was from 15 to 20 per 1,000. In Mauritius it was possible to compare Port Louis with and without sewerage and sewage disposal and the average figure was 17·6 per 1,000.

So far as Johannesburg was concerned the Bantu enjoyed the same type of sanitation as the European—that was the same for most urban Africans. The difficulty was the African in the Reserves. Slowly they were being educated to use sanitary buckets, the contents of which were buried in shallow trenches. The time would come when they would use the contents of the buckets for composting, but education was a slow process.

In Johannesburg a lot of farming was done. The last year the Author had been there, 28,000 bags of mealies or Indian corn, 20,000 bags of oats, and 5,000 tons of lucerne had been produced. In addition, grazing was let to farmers, who paid 3*d.* per head per day in the summer when they had grassland themselves and 4*d.* per head per day in the winter—which worked out at an average of 10*s.* a month per head. The income to the city for grazing cattle alone was £22,000 or £24,000 per annum. The Council had one of the finest herds of Herefords in South Africa. More than 90 per cent of the super-prime beef sold on the Johannesburg market came from cattle grazed on the sewage disposal works.

There had been a reference to the water-supply. Water was obtained from the Vaal River, which, without damming or without the barrage, would almost cease to run in the winter. The barrage threw back the water 40 miles, and the Vaal dam threw it back an additional 80 miles, and there was never less than 3 years' supply of water stored. Water was discharged from the barrage continuously and irrigated miles of country down the river, until it joined the Orange River. In addition, water was taken out of the river for the supply of water to the new Orange Free State gold mines. The water was pumped from the Vaal River to Johannesburg through a total head of nearly 2,000 feet. The Rand Water Board was a co-operative society; the local authorities guaranteed 46⅓ per cent of the total cost, the mines guaranteed 46⅓ per cent, and the railways 7⅓ per cent. Notwithstanding that it was being pumped 1,800 feet approximately, water was supplied to Johannesburg at 9*d.* per 1,000 gallons—a truly remarkable feat.

In reply to Mr Braine's comments, the Author observed that in subtropical countries land could take much more water, first by evaporation and then transpiration. Humus in the soil was destroyed very much more rapidly under tropical conditions and the "holding" content of water was determined a great deal by the humus content of the soil. During the dry season from April to September all digested sludge could be ploughed back into the land and it could be irrigated with a purified effluent. That improved the tilth of the soil and kept up its humus content.

What the Government were going to do with the water had not yet been determined. Between 60 and 70 per cent of the total water sold in any town in South Africa went back down through the sewer, and therefore there was a considerable quantity of water which would be available for re-use.

There had been a lot of trouble with concrete pipes. Some had lasted

for as short a period as 6 months. The South African Building Research Bureau seemed to be confirming the idea that the cause was mostly the sulphur bacteria. Only spun concrete pipes seemed to be effected—but spun pipes which had been supplied 30 years ago were as good today as when they were put in. Whether there was a difference in the quality or the chemical content of the cement, the Author was not yet in a position to say.

So far as he knew, no town having sewage had ever been rationed for water. Towns in Natal almost approached the American standard of 100 gallons per head per day.

The infiltration water did not seem to vary greatly, and amounted to about 1 cubic foot per minute per mile of sewer. With the ground temperatures in Mauritius, there was positively no infiltration. He did not know if he was correct in his theory, but he thought the fine hair cracks in the joints of the sewers were caused by the variation in temperature between the sewage and the ground, and there was unequal expansion and contraction. The ground temperature in Mauritius down to a depth of 10 feet did not vary more than about 3 or 4 degrees throughout the year and that appeared to be the reason that there was no infiltration. Further, there was no need to heat the sludge-digestion tanks.

On the subject of diet, the Author remarked that the Masai in East Africa lived on practically nothing else but emulsified blood and emulsified milk. Just as a blood donor was bled, so the Masai bled the ox for blood and he milked the cow for milk and emulsified them. Many doctors said that it was the best diet in the world ; but if people were looked upon as food factories, the waste products of the factories did in no uncertain way affect the design of the sewage-disposal works.

Mr Rawlinson had asked about the size of the city when the municipal development had started. In the original sewage-disposal works in Johannesburg, started in 1908, the sewage-disposal works had been designed to treat 1 million gallons per day. When the Author had been appointed to Johannesburg in 1927 the reconstruction of the works had started. That had meant the construction of four separate sewage disposal works in addition to the enlargement of the old works. The capacity of the latter had been increased to 60 million gallons per day. Three of the other works had been designed to treat 2 million gallons per day.

With regard to storm water, Johannesburg took a maximum of 4·63 inches per hour as the run-off, although he had sometimes seen 5 inches in 20 minutes. No storm water was supposed to be admitted to the sewer, but when a residential area was built on a slope and the streets were across the slope, there was a disinclination on the part of people to have water standing in their backyards, and they put the downpipe from the roof over the gully and drained the property. During storms as much as double the average daily flow was discharged into the sewer. Speaking generally, the executives in South Africa started work very much earlier in the day

than in Europe and the maximum flow was about three times the average flow, so that there had been as much as six times the average flow owing to storms. The storms usually came in Johannesburg between half-past three and six o'clock, which was just at the period of maximum flow.

The total money spent on recovery of sewage gas was for the provision of the collecting covers. That money would have been spent to collect the gas which would be required to heat the sludge. At the present time the gas was piped direct to the cyanide factory and the cooling water of the compressors installed in the factory was used to heat the sludge.

The Author had not heard of the sludge-freezing process. With an evaporation of 90 inches, as at Kimberley, and 70 inches, in Johannesburg, and when one was quite sure of having 290 fine days a year, a very thin layer of sludge did not take long to dry.

All local authorities had the right to charge for trade waste. Today, if a new factory wished to start, the Medical Officer of Health in Johannesburg would not give permission except on the report of the bio-chemist; if the bio-chemist said there had to be a payment, his decision was final.

Correspondence on the foregoing Paper is now closed and no further contributions other than those already received at the Institution can be accepted.—SEC. I.C.E.

THE DUGALD CLERK LECTURE, 1952 *

"Tunnelling Plant and Equipment"

by

George Charles Archer

Introduction

There are many types of tunnels, both large and small, which fall into the following categories:

Timbered headings for small pipes.
Tunnels in rock, lined and unlined.
Tunnels in soft ground.
Tunnels in waterlogged or silty ground.

These tunnels can be lined with:

Brickwork.
In-situ concrete.
Cast-iron segments
Pre-cast concrete blocks.
Pre-cast concrete bolted segments.
Pre-cast concrete expanded lining.

It will be appreciated that with so many alternative types of construction there is a great variation of plant and equipment in tunnel construction, and for the purpose of this Lecture I propose to confine myself chiefly to plant and equipment required for small-diameter tunnels, segmentally lined and constructed mostly for tube railways, main drainage, subways, and water, in soft ground such as London Blue Clay.

Sh aft Winding-Gear

Most tunnels are constructed from working shafts, and the lay-out of the equipment on the surface is of major importance if maximum tunnel progress is to be obtained.

The excavation and lining materials can be handled in two ways, either by crane or by headgear and cage, and in both cases it is essential for the equipment to be compactly laid out so that the time of each cycle of operation may be reduced to the minimum.

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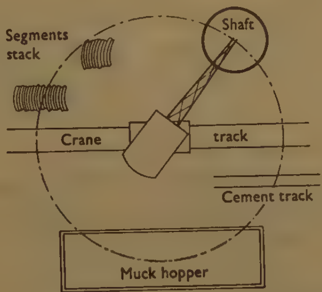
* This Lecture was delivered at a meeting of the Association of London Students on the 2nd January, 1952, and was repeated before the North-Western, South-Western, Yorkshire, Northern Counties, Glasgow and West of Scotland, and Northern Ireland Associations.

Cranes

All cranes used on shafts should be of the positive-gear type, such as steam or electric, and not frictional drive, but even then they should only be used in shafts of up to approximately 150 feet in depth, owing to the size of the winding drum and the number of "laps" involved by the winding rope. For shafts of greater depths a headgear should be used with a suitable winch provided with ample capacity for long lengths of winding rope.

Fig. 1 shows in diagrammatic form the relationship of a steam crane to a shaft and muck hopper. It is important that, so far as possible, the

Fig. 1



Typical Lay-out at Shaft Top using Crane

crane should wind out of the shaft and slew over the muck hopper, because any time used for travelling and luffing would be wasted. It should be borne in mind that there are approximately six loads of muck to one load of lining material. *Fig. 2* shows an actual lay-out.

There are, of course, occasions when tight grouping cannot be used, but the plant should always be laid out for the minimum of operations that is to say, winding, slewing, and travelling (but no luffing).

Fig. 3 shows a lay-out in a very restricted area. The whole of the plant on the site is built over temporary steel joists placed in position before the excavation of the chamber or shaft has been commenced. The crane track, muck hopper, and the lorry bringing in the segmental lining are all supported on these joists, and the entire new works are being constructed under this staging without further alteration of surface plant.

Headgears, Cage, and Muck-Hopper

Fig. 4 shows a steel headgear connected to a steel muck-hopper. In the construction of tunnels of more than ½ mile in length this type of headgear and cage is more efficient than a crane. The muck-hopper is kept as close as possible to the headgear, in order to save labour and time in dealing with the disposal of the muck. The headgear is fitted with a

steel cage, and the loading positions at ground and hopper level are maintained by the use of hinged brackets or "tumblers."

The cage is fitted with safety bars or gates, together with a roof to prevent accidents caused by falling debris, and railway tracks are laid so that skips can be run straight on or off at tunnel or surface levels. The guides or "leaders" of the cage are four wire bonds which are always kept taut and extend from the bottom of the shaft to the top of the head-gear. The hoisting is carried out by a suitable winch with positive gears and having speeds of between 100 and 200 feet per minute.

The steel muck-hoppers shown in *Figs 2* and *4* are constructed in prefabricated sections and can be built to any length in multiples of 5 feet. For transport purposes it has been designed in such a way that the component sections of a 40-foot-long-by-12-foot-wide muck-hopper can all be loaded on to one lorry. The capacity of the muck hopper should be sufficient for 24 hours of maximum progress, to obviate night cartage.

Digging Grabs

Single-chain digging grabs of ½ to 1 cubic yard capacity are extensively used for grabbing the excavation from shafts through soft ground, particularly those sunk by the open caisson method. See *Fig. 5*.

Pressure-Grouting Pans

Fig. 6 shows a typical bougie or pressure-grouting pan. This apparatus is used for forcing grout under pressure around the outside of the segmental lining. It is a steel cylinder with a screwed-down lid at the top. Passing through this cylinder is a steel spindle with mixing blades rotated by a power-driven pneumatic motor. In operation, the pan is half-filled with water, bags of cement are placed on the tray and gradually emptied into the pan, while the blades are rotated until the grout is of a thick creamy nature. The lid is then screwed down and compressed air is applied into the top of the pan forcing the grout through the bottom pipeline and the flexible rubber pipe to the "gun" screwed into the segmental lining. When the pan is emptied, the "gun" is closed, the air in the pan exhausted, and the whole operation repeated. The pan is fitted with an air by-pass line so that the flexible pipe and "gun" can be blown out; also, by kinking this flexible pipe the air can be blown back into the pan should the main valve become blocked. Where grouting pans have no power-driven motors, the spindle has a cranked handle for manual operation. The usual capacity of a pan of this type is either 4 or 9 cubic feet.

Pneumatic Tools

Fig. 7 shows various types of compressed-air tools used in tunnel work. Reading from left to right: a rock drill; a heavy concrete or road

breaker; a clay spade; a clay pick; a Quimbey air-pump; a sludge-sump pump; a rotary machine of the type used either for boring wood or for drilling steel; a caulking machine; and an internal vibrator.

The clay spade can, of course, be fitted with any width of blade, according to the hardness of the clay.

In rock tunnels, "drifters" or heavy drills can be used either with pillar support or from a drilling carriage, but this type of machine is steadily being replaced by the rock drill fitted with a self-adjusting pneumatic-feed leg (*Fig. 8*). The latter is lighter in weight and therefore easier to handle, and although the drilling speed is slightly less than that of the "drifter" its consumption of compressed air is approximately 50 per cent less.

High-Pressure Compressors

High-pressure compressed air, with a working pressure of 100-lb. per square inch, is now generally used for many types of plant and equipment in tunnel work, particularly for the small tools, hoists, pumps, conveyor motors, loaders, etc. It is therefore essential for an adequate supply of compressed air at the correct pressure to be maintained on the surface, and stand-by compressors should be available for immediate use.

I would stress here that slow-running compressors, either diesel or electrically driven, are to be preferred, and an ample margin of power should be allowed for. For instance, the specified air-consumption of a machine tool will probably be found to be greatly increased or even doubled after it has been in continuous use for 3 or 4 weeks, and so most of the small air-tools, such as clay spades, if used on a 24-hour basis, lose a great deal of their efficiency after 6 months and should be renewed.

The compressed air is taken to the face by pipeline, the size of which varies according to the magnitude of the tunnelling operations and the amount of compressed-air equipment being used, but should never be less than 2 inches in diameter. If the tunnel is not large enough to install small air-receivers in order to maintain a good balance of air pressure, short lengths of, say, 40 or 50 feet of larger-diameter pipe can be incorporated in the air line every 200 or 300 yards.

Pumps

There are various types of pumps which can be used for sinking shafts, the four most common being:

steam pulsometer;
electric;
compressed air; and
rocker arm.

Fig. 9 shows an efficient shaft-sinking pump which can be lowered as the work proceeds. It is electrically driven, fitted with flexible delivery

pipes, and is suspended by wire bonds, the length of suction thus being constant.

It must be borne in mind that pumps lose a high percentage of efficiency if their suction head is greater than 15 feet.

The types of pumps used in the construction of tunnels are:

reciprocating air or steam;
electric centrifugal; and
compressed air.

Diesel or petrol prime-movers should not be used for pumps in tunnels because of the danger caused by exhaust fumes.

Self-priming pumps are a great advantage at all times, and it is important to allow for pumps of a larger capacity than is thought really necessary.

Tunnel Shields

The purposes of the tunnel shield are to support the roof and sides in soft ground without timber, to enable erection of the segmental lining, and to facilitate tunnelling in bad ground.

All shields are basically the same, comprising a cylindrical steel skin with hydraulic rams for pushing the shield forward. At the leading end there is a cutting edge which can be provided with or without a hood, according to the strata for which the shield has been designed, and at the rear, a tail, inside which the segmental linings are erected.

Fig. 10 shows a "skeleton" shield with hood for loose ground. This shield gives the maximum possible working room, having an internal diameter only 2 feet 6 inches smaller than the outside diameter. This type of shield is used mostly for free-air work because if used in compressed air and a "blow" occurred, water and silt would have free and uninterrupted passage into the tunnel.

Fig. 11 shows another type of shield incorporating a diaphragm which reduces this danger. Through the diaphragm there is an opening to give access to the face, the top half of which can be closed with a steel hinged door, and the bottom half with dam boards which are placed in the channel grooves provided, thus sealing off the workings.

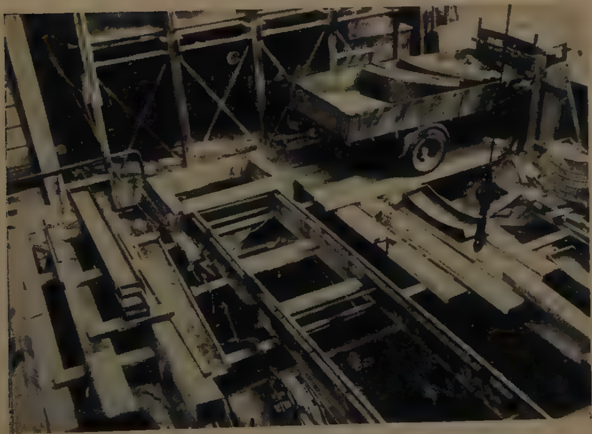
Some shields, used exclusively in compressed-air work in very bad ground, are fitted with a water trap, as will be seen in *Fig. 12.* When a "blow" occurs, the working in front of the diaphragm fills with water and so forms a water seal which enables the air pressure in the tunnel to be built up without further loss at the face.

Occasionally, shields used for tube railway tunnels in London have been fitted with rotary diggers. *Figs 13* shows the front of one of these diggers; the cutting knives and scoops are rotated by an electrical motor and reduction gear fitted in the body of the shield. This type of digger shield, in dry ground such a London Blue Clay, is capable of maintaining a very high rate of progress.

Fig. 2

ACTUAL LAY-OUT AT SHAFT TOP USING CRANE

Fig. 3



SURFACE LAY-OUT AT A VERY RESTRICTED SITE

Fig. 4

Steel Headgear and Steel Muck-hopper

Fig. 5

1-cubic Yard Digging Grab

Fig. 6

Typical Pressure-Grouting Pan

Fig. 7

Typical Compressed-air Tools

Fig. 8

Pneumatic Feed Legs for Rock Drills

Fig. 9

Electrical Shaft-Sinking Pump

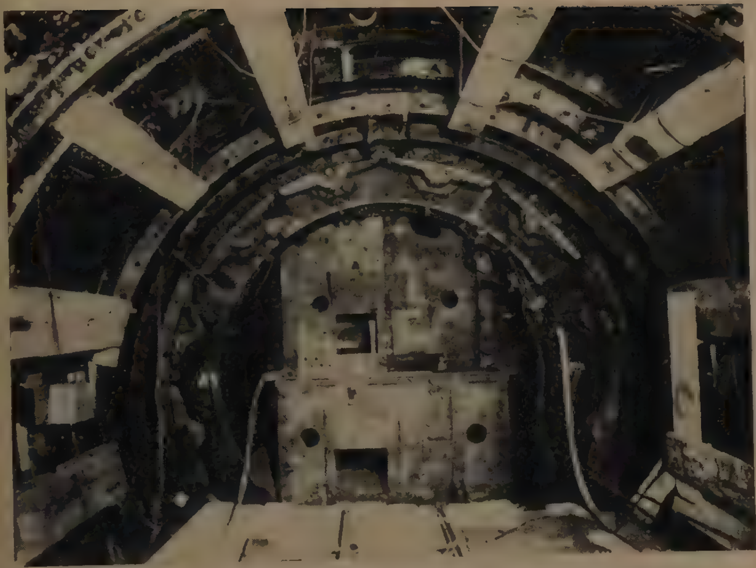
Fig. 10

"Skeleton" Shield with Hood for Loose Ground

Fig. 11

Diaphragm Shield with Door

Fig. 12



Diaphragm Shield with Water Trap (in shield chamber)

Fig. 13

Rotary Digger Shield (front view)

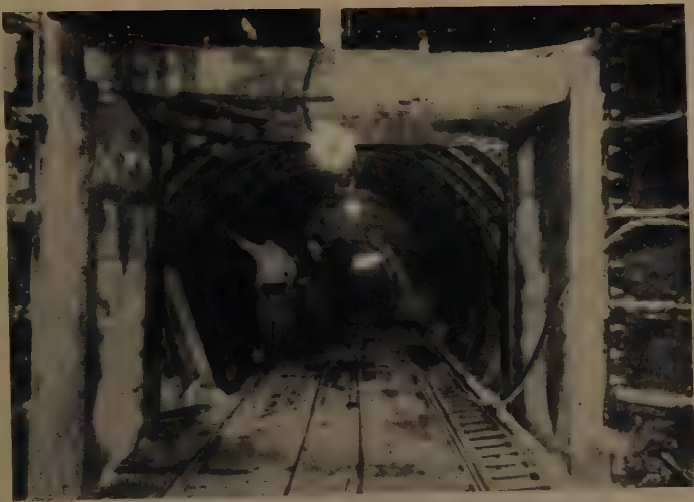
Fig. 15

LOWERING SHIELD (VERTICALLY) DOWN SHAFT (NOTE SMALL CLEARANCES)

Fig. 16

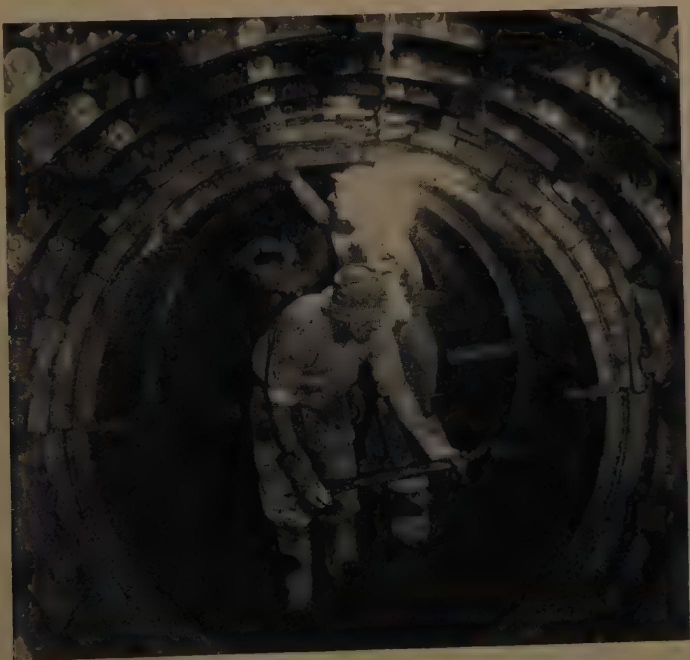
MECHANICAL MUCK-LOADER

Fig. 17



SHORT LENGTH OF LARGER TUNNEL AT SHAFT BOTTOM FOR MARSHALLING TRAFFIC FOR SMALL TUNNEL CONSTRUCTION USING LARGE ROLLING STOCK

Fig. 18



LINING EQUIPMENT FOR IN-SITU CONCRETE

Fig. 19

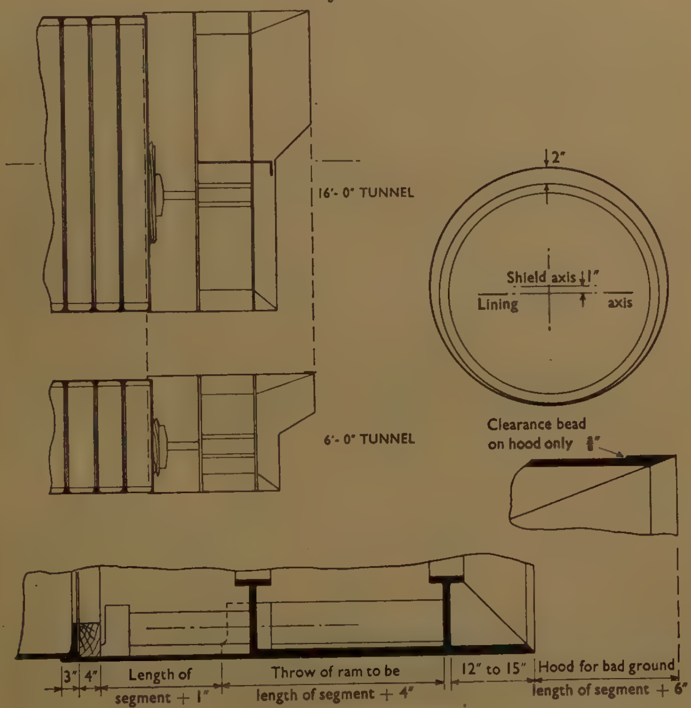
Low-Pressure Compressed-air Lay-out for 9′ 0″ Tunnel

Fig. 21

Typical Vertical Air-lock in Use

Figs 14 show in diagrammatic form that a shield for a 16-foot-internal-diameter tunnel is the same length as one for a 6-foot internal diameter. In designing a shield, particularly of small diameter where the diameter is less than the length, it is very important to keep the shield as short as

Fig. 14



TUNNEL SHIELD DESIGN

possible, because the smaller the diameter compared with the length the more difficult is the shield to control and to keep to line and level. In its design, minimum allowances should be made, as follows :—

(1) The tail should have a 3-inch hold on the last ring of lining, when the shield is fully advanced.

(2) Length of tail should allow for length of segments plus one inch clearance, plus 4 inches for a grouting rib between the segments and the feet of the rams.

(3) The hydraulic rams should have a "throw" 4 inches greater than the length of the segment (this will give room for packing out the segmental lining on curves).

(4) When the shield is being designed for use in bad ground, the hood should be 6 inches longer than the length of segment, since no

excavation should take place beyond the cutting edge. Th cycle of operation must be one "shove" per ring of segmenta lining.

(5) The inside diameter of the tail should be 2 inches greater tha the outside diameter of the segmental lining, to enable th segments to be "keyed-in."

(6) The number of hydraulic rams required, except in special cases is taken as the tunnel diameter in feet. For example, a shiel 8 feet in diameter would have eight rams; one 12 feet 3 inche in diameter would have twelve rams.

Since the segmental lining is built up at the tail at invert level, ther will be a 2-inch "keying-in" space in the soffit; therefore, the axis o the segmental lining is always 1 inch lower than the axis of the shield.

Tunnel shields are always made in segmental sections, bolted an studded together, and are generally built in shield chambers constructe specially for their erection, an operation that is made particularly difficul by the restricted working space in the chamber. (See *Fig. 12.*)

In recent years, complete shields have been lowered vertically down th shafts into shield chambers built at their base. *Fig. 15* shows an exampl of this operation, which affords a great saving of time and money.

Tunnel Loading

Generally, excavation in tunnel work is loaded by manual labour, bu in rock tunnels mechanical loaders are extensively used. An example of this type of machine, driven by high-pressure compressed air, can be see in *Fig. 16.*

Conveyors are also used for loading the excavation behind tunn shields, but for easy marshalling of traffic these should extend back in the tunnel for a length equal to a full train of skips.

Rolling-Stock

The most commonly used gauges for rolling stock are 24-inch and 1 inch although in very small tunnels 15-inch gauge may be used wi advantage.

The skips, which vary in capacity from ½ to 1 cubic yard, should alwa be of U-shape side-tipping design, thus enabling two skips to pass ea other in the smallest possible width of tunnel and also facilitating unloadin

On all short tunnels, the skips are pushed manually from the worki face to the shaft bottom but, where the distances are long, electrical loc motives of the battery type are frequently used.

It is good practice to use 24-inch gauge with the larger capacity rolling stock wherever possible, for this will greatly reduce the number trips, and even in smaller tunnels it is often advisable to use the large rolli stock. If the drives are long, short lengths of larger-diameter tunn

an be constructed on each side of the working shaft and at intervals of 00 to 600 yards to give marshalling and passing room for all traffic. See *Fig. 17*).

Lining Equipment

Apart from cable tunnels or Tube Railways, most tunnels, and especially hose for main drainage, are faced inside the segmental lining either with n-situ concrete, brickwork, or both. In all cases the method of construct-on is similar and consists of steel ribs and timber laggings which have " block " laggings at right angles between them in the soffit. Where the n-situ lining is 3 inches or 4 inches thick or a single ring of brickwork, he " block " laggings should be 15 inches wide to enable the man " keying-n " to pass his head through to carry out inspection. A typical example of this can be seen in *Fig. 18*. In single-brick lining, the laggings should be only one course of brickwork wide, so that they can be fixed, course for course, in the correct position and the brickwork laid behind.

Low-Pressure Compressed-Air Equipment

Where the ground is unstable, such as running sand, and cannot be tunnelled by ordinary methods, low-pressure compressed-air may be employed.

The compressed air is retained in the tunnel or shaft by bulkheads, in which are incorporated air-locks to enable workmen and materials to pass in and out of the workings.

To supply this low-pressure air, units of large compressors are installed on the surface, and particular attention must be given to the lay-out of this plant. Once the compressed air is put into operation it cannot be taken out of the workings safely until completion, except by using very expensive precautionary methods.

It is advisable to provide for the maximum requirement of compressed air by a series of small units so that the supply can be varied economically according to the demand. If the prime movers are electric motors, then 100-per-cent stand-by should be installed with alternative means of power. If the prime movers are diesel, then the stand-by capacity need only be in the region of 50 per cent. *Fig. 19* shows a lay-out for a 9-foot-diameter tunnel (single face) and illustrates the magnitude of low-pressure compressed-air equipment. In addition, air cleaners and coolers can be installed, if required.

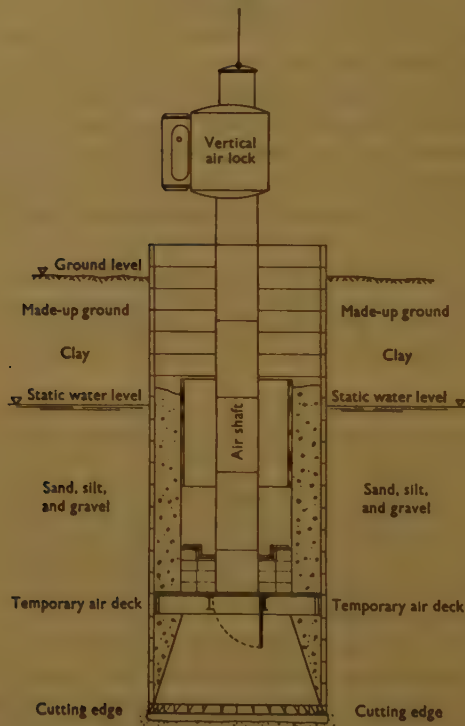
Vertical Air-locks

The sinking of shafts necessitating the use of compressed air, when constructed by either the underpinning or caisson method, requires similar equipment comprising a horizontal air deck fitted with vertical tube and air-lock. (See *Fig. 20*.)

All locks are composed of a chamber with two doors, one of which kept closed by the air pressure.

Fig. 21 shows a vertical air-lock; the muck-lock is immediately ove the shaft, and the man-lock is in the form of a " blister " attached to th side of the main chamber. The Figure shows a segment being lowere into the muck-lock, and it will be seen that the bond of the crane passe

Fig. 20



TYPICAL SHAFT SUNK AS A CAISSON SHOWING VERTICAL AIR-LOCK, TUBE, AND DEC

through a gland in the detachable lid or door. The lid is fastened to th top of the air-lock by clips and two automatic air pistons, thus ensurin that the lid is retained in position and kept airtight.

The vertical air-lock tube which extends between the vertical air-loc and the deck should be of " figure 8 " section. The muck-skips and materi pass up and down the large section of the " 8 ", whilst the men have acce by a steel ladder in the smaller section, thus separating men and materia and so eliminating any danger of workmen being trapped whilst proceedi up or down this tube.

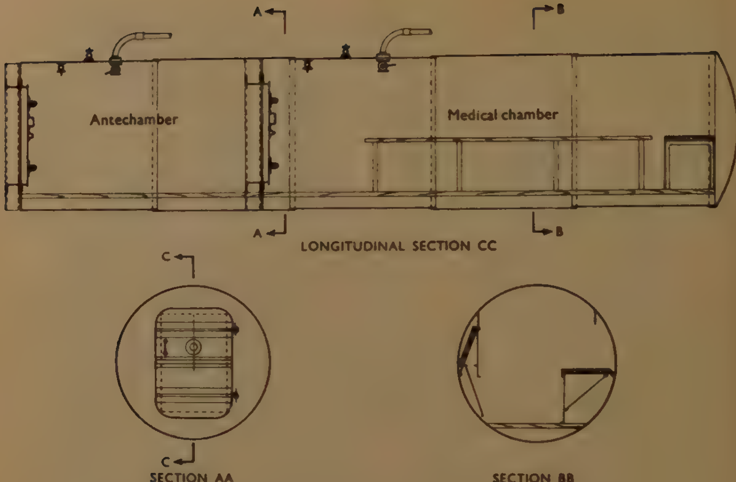
Fig. 22

TYPICAL HORIZONTAL AIR-LOCK ("BOILER" TYPE)

Fig. 23

CONCRETE BULKHEAD AND DOOR FOR HORIZONTAL AIR-LOCK IN SMALL TUNNELS

Fig. 24



Typical Arrangement for a Medical Air-lock

Fig. 25

Medical Chamber Ready for Use

Horizontal Air-locks

In the construction of tunnels necessitating the use of compressed air, bulkheads of either brickwork or concrete are built and through them a boiler-type horizontal lock is fitted, as shown in *Fig. 22*, although in smaller diameter tunnels, bulkheads with airtight doors are constructed so that the tunnel becomes part of the air-lock itself (*Fig. 23*).

All services required for tunnel construction, including low- and high-pressure air lines, water, electricity, telephone, hydraulic pipes, snorers, and additional spare pipes must pass through these bulkheads. The air-lock itself is operated by a special two-way valve which can fill or empty the air-lock.

All low-pressure air pipes should be fitted with a non-return flap on the pressure side of the bulkhead; this will eliminate the danger of large quantities of air escaping from the tunnel or shaft should the delivery pipes be damaged or broken between the bulkhead and compressor house on the surface.

Medical Lock

Men working in pressures of approximately 18-lb. per square inch or more are subject occasionally to " compressed-air sickness " when decompressed to atmospheric pressure. The only immediate relief of the pains caused by this sickness is obtained by replacing the patient in compressed air, and a suitable medical lock must therefore be available on the surface so that such cases can receive immediate treatment. *Figs 24* shows a typical arrangement. The lock is similar in shape to a boiler and is divided into two chambers, each fitted with an airtight door; one is a medical chamber containing the beds (*Fig. 25*) used for treatment of the cases, and the other an antechamber used for passing through to the medical section without interfering with the air pressure inside. It is common practice to fit a small food-lock in the medical chamber so that food, etc., can be passed through without wastage of compressed air. The medical lock should also be fitted with electric light, heat, telephone, and small inspection windows.

Conclusion

Owing to lack of time no mention has been made of such equipment as concrete mixers, ventilation, telephones, electric lighting, etc., the provision of which must be carefully thought out. The foregoing, however, will no doubt give a comprehensive picture of the kinds of plant used in tunnel construction.

ELECTION OF ASSOCIATE MEMBERS

The Council at their meeting on the 24th June, 1942, in accordance with By-law 14, declared that the undermentioned had been duly elected as Associate Members.

Home

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CORRESPONDENCE

On a Paper published in Proceedings Part I, January 1952

Paper No. 5788

"Anchored Sheet-Pile Walls".†

by

Peter Walter Rowe, Ph.D., A.M.I.C.E.

Correspondence

Professor G. P. Tschebotarioff, of Princeton, New Jersey, congratulated the Author on his comprehensive study which had achieved an important step forward in its field. The Professor was gratified that under identical test conditions, but using different measurement procedures, the Author had obtained results almost identical to his own. Thus the reliability of the novel bending-strain-measurement techniques employed by the Professor had received additional confirmation. His findings concerning the limited effectiveness of "arching" of sands had also been substantiated. However, the Author had suggested that the Professor's design recommendations concerning fixity conditions below the dredge line (*Fig. 1* (*e*)) were not justified for stiff piles in very loose sand (*Figs 16*). Professor Tschebotarioff conceded that point, but made the following further comments:

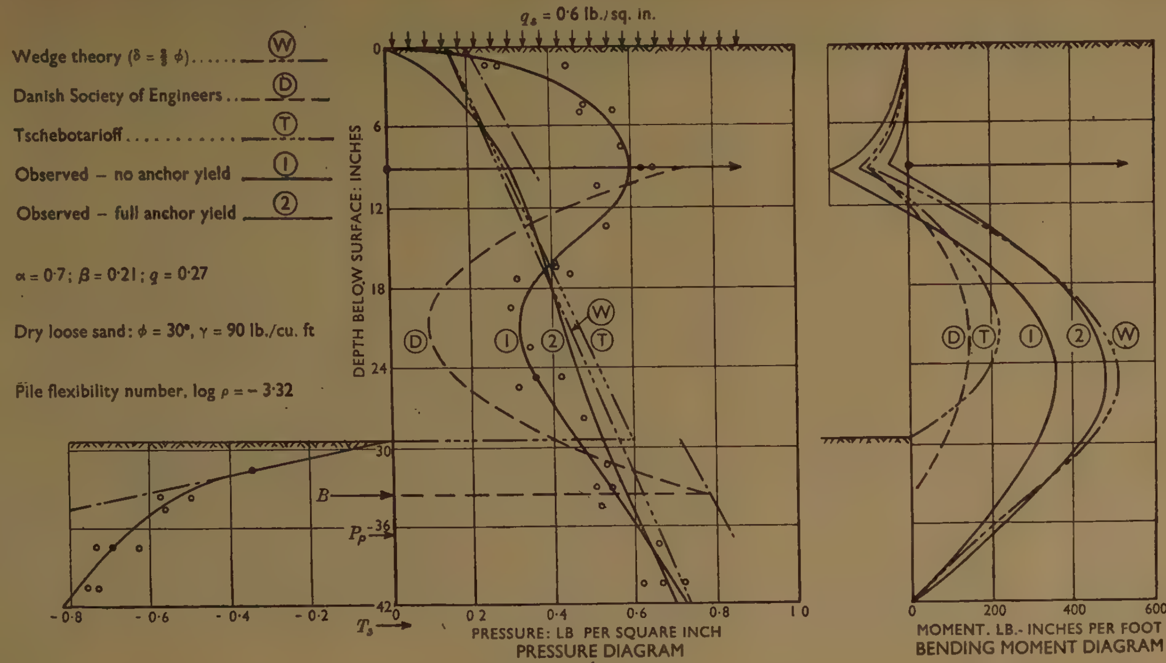
(1) According to *Figs 7 and 8*, Method (T) gave bending moments smaller than Method (D). That was an error caused by placing point B at two-thirds of the *actual depth* of embedment. The Danish Rules required point B to represent the location of the passive earth resistance P_p for the *calculated depth* of embedment, whereby P_p was determined [12] for a factor of safety of unity and an angle of wall friction $\delta = \frac{3}{4}\phi$ or $\delta = \frac{1}{2}\phi$ (1937 Rules). The actual depth was made greater than the calculated depth.[13] By applying to *Figs 8* the 1937 Danish Rules the correct point "B" was obtained (see *Figs 27*). *The maximum bending moment* (D) *was smaller than* (T) *by at least 25 per cent.* That was why

† Proc. Instn Civ. Engrs, Part I, vol. 1, p. 27 (Jan. 1952).

[12] See reference 1, p. 70.

[13] I. A. Rimstad, "*Zur Bemessung des doppelten Spundwandbauwerkes*" ("The Analysis of Double-Acting Sheet-Pile Retaining Walls"). Dan. Soc. Civ. Engrs 1940.

Figs 27



CORRECTION OF *Figs 8*

(The Rules of the Danish Society of Engineers (Method D) give a smaller value of the maximum bending moment as compared with Method T.)

Professor Tschebotarioff had not specifically excluded stiff reinforced concrete piling from his design recommendations (T), although his own tests had not included models thereof with log $\rho < -2{\cdot}97$; such piling *had* been designed according to the Danish Rules and had withstood the test of practical experience. The large difference between the (D) and the (1) and (2) moment values of *Figs 27* might mean that :—

(*a*) The factors of safety provided by the Danish Rules were even smaller than the Professor believed [14];

(*b*) Sands of such extremely low density as in that test did not occur in nature.

A combination of both factors was likely since, if (*a*) alone were valid failures in bending of Danish bulkheads should have become known. It was therefore doubtful whether such loose sands (at zero relative density) could actually be encountered, especially after sheeting had been *driven* into them.

(2) The Author used Stroyer's moment reduction factor,[15, 16] which was governed by the ratio: $\dfrac{\text{wall-thickness}}{\text{span}}$. For equal values of L and E the thickness of a stiff concrete wall with log $\rho = -3{\cdot}49$ should be roughly three times that of a flexible wall with log $\rho = -2{\cdot}07$. The corresponding moment increase for the stiff wall recorded in *Figs 1*… was of the order of 250 per cent, whereas the change in Stroyer's moment reduction factor had one-tenth of that value. Thus the test date presented did *not* support an implied general numerical agreement with Stroyer's values.

(3) Professor Tschebotarioff's large-scale tests had *not* been limited [17] to fixed earth support conditions, in spite of a contrary implication on p. 29.

(4) Actually, the contradiction between Browzin's and Professor Tschebotarioff's experiments mentioned on p. 29 was caused by the following. Browzin's piles had varied from $H = 9{\cdot}5$ to $H = 14{\cdot}2$ inches length. Naked-eye observations through a side glass plate had been substituted for precise measurements. Slight reverse curvatures indicating partial fixity could not be detected in that manner, especially at such a small scale; the point was of importance for future research. Contradicting Browzin's assumption, the Professor's measurements showed that such fixity might be present although the pile toe moved forward.

(5) The Author's method of "operating" and "structural" free earth support curves was a very useful new tool for research and design with

[14] See reference 4, p. 70.

[15] See reference 11, p. 70, *Fig. 30*.

[16] R. Stroyer, "Concrete Structures in Marine Work." Knapp, Drewett & Sons Ltd, London, 1937. *Fig. 20*.

[17] See reference 4, p. 70, test No. 57F ($\alpha = 0{\cdot}89$) and others.

special application to stiff piling and soft soils—for example, sand-clay mixtures about which much had yet to be learned.[18] However, no single design method could correctly reflect all details of possible field conditions. Fixed earth support of bulkheads frequently was a reality and should be used accordingly in design.[19, 20] Space restrictions precluded a more detailed discussion.

Professor Karl Terzaghi, of Cambridge, Massachusetts, observed that the Author had presented, for the first time, quantitative information concerning the influence of the flexibility of sheet piles on the maximum bending moment in the piles. He had also produced conclusive evidence for the influence of the yield of the anchorage on the distribution of the active earth pressure on dredge bulkheads. His findings were so important that they would have a lasting influence on methods of bulkhead design. The following comments referred only to details which had no bearing on the intrinsic value of the Paper.

On p. 33 it had been stated that the active pressure on the wall, prior to the beginning of the outward movement of the wall, was equal to that "given by the Coulomb theory with no wall friction." That was a mere coincidence, because the major part of the wall friction was active before the wall started to yield. Furthermore, the Author had assumed throughout his Paper that a yield of $H/1{,}000$ was sufficient to reduce the lateral pressure of loose sand to the Coulomb value at full wall friction, and that an additional outward movement of the wall had no effect on the lateral pressure of the sand. According to Professor Terzaghi's observations the lateral pressure of loose sand decreased very conspicuously until the yield became equal to about $H/5{,}000$. Additional movement was associated with a slow but steady further decrease of the lateral pressure, and the lateral pressure did not assume its minimum value until the wall had moved to a distance considerably in excess of $H/200$.

The preceding comments were based on the results of large-scale tests performed by Professor Terzaghi in 1929. Those tests had furnished accurate values for both the horizontal and vertical components of the lateral pressure of loose sand with an angle of internal friction of 34 degrees for different values of the lateral yield of the wall and, as a consequence, it had been possible to compute the values of the angle of wall friction δ and the mobilized part ϕ of the angle of internal friction for different values of lateral yield.[21] An abstract of the test results was given in Table 5 in

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[18] G. P. Tschebotarioff, "Some Unsolved Problems of Importance for the Design of Earth Retaining Structures." Perm. Inter. Assocn Nav. Congr., Bull. No. 33, Brussels, 1950.

[19] Hermann Blum, "*Beitrag zur Berechnung von Bohlwerken.*" W. Ernst u. Sohn, Berlin, 1951.

[20] G. P. Tschebotarioff, "Soil Mechanics, Foundations and Earth Structures." McGraw-Hill Civ. Engng Series, New York, 1952.

[21] Karl Terzaghi, "Large Retaining-Wall Tests." *Engng News Record*, vol. 112 (1934), *Fig. 2* on p. 137.

which K_A denoted the coefficient of earth pressure, δ the angle of wall friction computed on the basis of the measured values of the vertical and horizontal earth-pressure component, and ϕ the mobilized part of the angle of internal friction.

TABLE 5

Yield	K_A	δ	ϕ
0	0·405	21° 20′	19° 30′
$H/25,000$	0·371	26° 0′	20° 50′
$H/7,000$	0·320	25° 30′	25° 10′
$H/1,200$	0·279	26° 40′	28° 40′
$H/200$	0·247	26° 20′	32° 20′

Table 5 showed that the development of the wall friction preceded the development of the internal friction. At a yield of $H/200$ the value of K_A had still been decreasing at constant wall friction which indicated that the internal frictional resistance of the sand had not yet been fully mobilized. In contrast to the performance of loose sand, and in agreement with the Author's observations, the lateral earth pressure of dense sand had assumed its minimum value (Coulomb value at full wall friction) at a yield of less than $H/1,000$ and further yield of the wall had been associated with a steady increase of the lateral pressure. Vibrations had reduced simultaneously the angle of wall friction and the active part of the internal friction in both loose and dense sand.

On p. 37, the Author had listed the conditions for similarity between model and prototype. To them should be added the condition that the ratio between horizontal displacement of the sheet piles and the corresponding horizontal soil reaction should depend only upon the relative density of the sand. In reality it also depended to some extent upon the scale, which caused an error in the interpretation of the test results. However, the error appeared to be on the safe side.

On p. 60, item (2), the Author had stated that a "considerable shear force acts at the toe of the pile at this stage, owing principally to the vertical component of the active pressure." That horizontal force T_s (p. 65) was equal to the resultant of all the vertical forces, such as the weight of the pile and the wall friction which acted on the pile, multiplied by the coefficient of friction between the toe of the pile and the sand. The resultant vertical force produced a downward movement of the pile. Even an imperceptible downward movement of the pile almost eliminated the wall friction on the active side and increased the wall friction on the passive side. The movement stopped as soon as the resultant vertical force became equal to the point resistance of the sheet pile. In dense sand the point resistance was very small and in loose sand it was negligible.

For that reason it appeared that the shear force at the toe of the pile did not deserve any consideration.

The design charts, *Figs 20* and *21*, had been plotted on the assumption that the relative density of the sand into which the piles had been driven did not change with depth. In practice that condition was very rarely satisfied. As a rule the density of sand changed with depth in an erratic manner. On a pile-driving job on Vancouver Island, the depth at which the piles had met refusal in what had appeared to be a remarkably uniform sand deposit had varied within a single pile cluster between 62 and 82 feet. Above the lenses of dense material the sand had been so loose, that the number of blows per foot of penetration had been consistently less than five and within the lenses it had increased to thirty or forty. Similar conditions had been disclosed by cone-penetration tests in a deep sand stratum on Houston Street in New York and at various other localities. Hence a direct application of the Author's charts to design problems would not be indicated unless the subsoil explorations demonstrated beyond any doubt that the relative density of the sand into which the sheet piles would be driven was really more or less independent of depth. Professor Terzaghi's experience with sand strata suggested that that condition was very seldom satisfied. Furthermore, on large jobs, it was impracticable to ascertain the local variations of the relative density of the sand accurately enough to eliminate the risk of serious misapplication of the charts. Hence the instances in which the charts could safely be used as a basis for design were rather rare. However, the charts were an admirable means of illustrating the decisive influence of the flexibility of the sheet piles on the maximum bending moment and disqualifying the methods of computations which were based on the assumption that the conditions of end support depended only on the depth of sheet-pile penetration.

If the locks of a row of sheet piles were located in the neutral axis the flexural rigidity of the bulkhead depended to a large extent on lock friction. Since the lock friction could not be reliably ascertained by any practicable means in advance of construction, the computation should be made on the two extreme assumptions that the locks were frictionless and that the locks performed like welded joints.

The value of the Paper would be greatly enhanced if the Author, in his reply, would make definite statements concerning the units which he used in his equations such as those on p. 60. Otherwise the equations could be misleading, because they were dimensionally incorrect.

Mr Savile Packshaw and **Mr J. Owen Lake**, in a joint contribution, observed that the Author had made an outstanding contribution to present knowledge of the behaviour of flexible retaining walls and that his research, even though carried out on a small model and with limited resources, would have a marked influence on the design of all types of sheet-pile structures.

It had long been known that the bending moment in an anchored flexible

wall was smaller than that derived from a calculation based on the classical earth-pressure theories of Coulomb or Rankine. That had been proved experimentally by Stroyer[22] and was generally ascribed to "arching," the assumption being that, if the wall was supported more or less rigidly by the tie rods at the top and by the embedment of the pile in ground at the bottom, there was a redistribution of pressure which led to a concentration of pressure at the supports and a reduction at mid-span. That, in turn, led to a reduction in bending moment.

Stroyer's research had been confined to measuring the bending moments induced in his model because at that time no satisfactory earth-pressure gauge had been developed. The successful design of such a gauge was difficult because it had to measure intensities of pressure over a wide range without requiring a movement or a deformation of more than a few thousandths of an inch; otherwise the soil might arch across the gauge and false readings would then be recorded. The Author's work had largely been made possible by the use of a type of pressure gauge which he himself had developed.[23]

The Author's research could be divided into two parts. In the first, he had measured the intensities of active and passive pressures on the wall, using a section which, in the light of his subsequent research, could be described as rigid. In the second part he had investigated the relation between the flexibility of the wall and the bending moment induced in it; intensities of earth pressure had not been measured. The conclusions which he had reached from the first portion of his work were summarized on pp. 33 to 36 and illustrated by *Figs 7* and *8*. They ranged from a wall with tie-rods at the top and no superload to a wall with tie-rods at a relatively large depth and a very substantial superload. Among the points of interest were :—

1. The Author's bending moment curve 1, for no tie-rod yield—for example, in the case of a wall fixed to a rigid relieving platform supported on bearing piles and prevented from moving laterally by raking piles—corresponded closely to curve S for the bending moment derived from the Rankine theory after application of Stroyer's reduction factor (p. 62). In other words, that series of tests confirmed that under the particular condition of no yield at anchor level there was a redistribution of pressure as shown by curves 1 of the pressure diagram. That was caused by arching and by the influence of the underlying semi-rigid boundary created by the soil below dredge level.
2. The Author had found that a tie-rod yield of $H/1{,}000$ was always sufficient to destroy arching and boundary effects. The elastic extension of tie-rods stressed to, say, 7 tons per square

[22] See reference 2, p. 70. [23] See reference 6, p. 70.

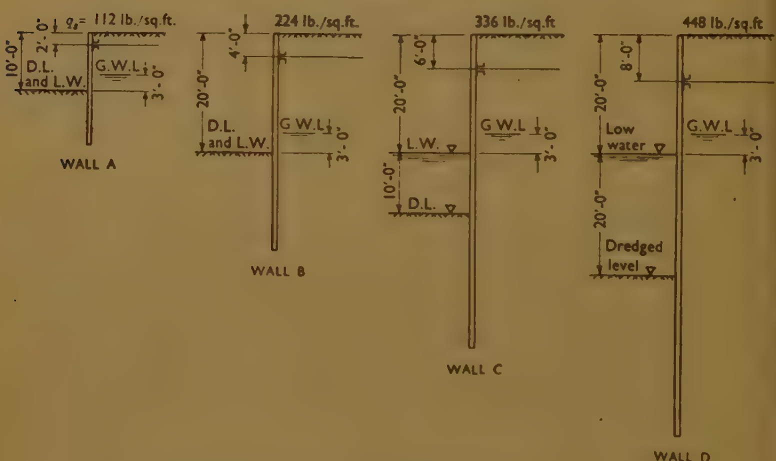
inch on the net area under the thread (and substantially less on the gross area of the plain rod) would generally be much less than $H/1{,}000$. However, allowance had also to be made for the consolidation of the ground in front of the anchorages and it was therefore generally reasonable to accept the Author's opinion that the tie-rods should be regarded as yielding supports. However, where the tie-rods could be pre-stressed by tightening before dredging or excavation brought the active pressure into action, it might be permissible to design the wall on the assumption that the tie-rods were unyielding supports.

3. The passive-pressure distributions in *Figs 7* and *8* showed that, for a stiff wall of normal penetration with a retained height of $0{\cdot}7H$, only part of the potential resistance had been mobilized and that no full wall friction had developed. The Author's opinion appeared to be that wall friction did not develop until failure of the passive wedge was imminent and that seemed reasonable. The Author's conception of a horizontal shearing force T_s acting at the toe of the wall in addition to the passive pressure P_p also appeared to be logical, provided it was understood that the reaction T_s acted in a backward direction only under conditions of free earth support (*Fig. 15* (*a*)). For a fixed wall, T_s would act in the opposite direction. It should be noted that the Author's percentage reductions of bending moments (*Figs 13* and *14*) were related to the moment induced in stiff piles and that justified the backward direction which he had assigned to T_s. The fact that full wall-friction could be developed in dense soil was shown by *Fig. 15* (*b*), in which an angle greater than $\frac{2}{3}\phi$ had been obtained for a rotation of only $0{\cdot}005$ radian, equivalent to a forward movement at ground level of, say, only a little more than $\frac{1}{2}$ inch for a penetration of 10 feet.

With regard to the Author's research on the relation between bending moment and flexibility, that was clearly illustrated by *Figs 16*. The conventional method of design generally in use at present assumed that the wall would be "fully fixed" and that its deformation below dredged level would be such that it could, fortuitously, develop the resistances required for the formation of negative bending moment. Another method was to design the wall to be "freely supported," the penetration then being increased to ensure an appropriate factor of safety. That method led to shorter piles but required a heavier section owing to the greater bending moment and, as a rule, it was not favoured because it possessed a smaller margin of safety against unexpected superloads or excessive dredging. It was apparent from *Figs 16* that full fixity (that was, a negative bending moment almost as great as the positive) was always developed when the wall was subjected to the pressure of loose soil but penetrated into dense

soil. It was very probable that nearly the same results would be obtained if the soil were dense throughout the whole height of the wall. On the other hand, only a small degree of fixity had been obtained with loose soil, even though there had been some negative moment, however slight, in every case. Bearing in mind that the Author's research had been carried out entirely on non-cohesive soils and that loose non-cohesive soils were uncommon in their natural state, it could be said that most walls designed by the conventional method to be fixed in non-cohesive soils really developed the required negative moments. That would doubtless be a source of satisfaction to their designers.

Figs 28



Soil properties :

Loose sand: Angle of internal friction, $\phi = 30°$. Natural soil density, $\gamma = 100$ lb./cu. ft. Submerged soil density, $\gamma_s = 60$ lb./cu. ft.

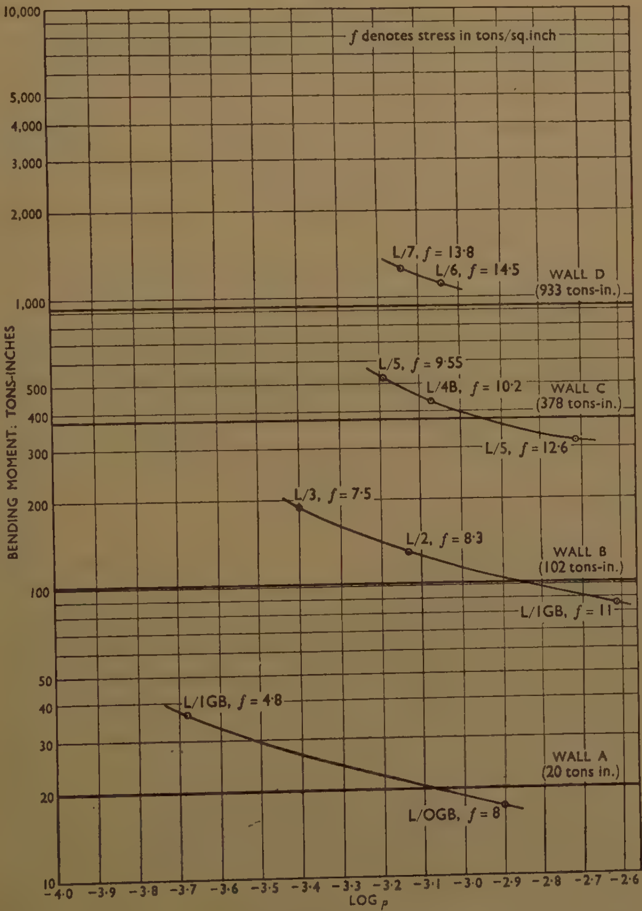
Dense sand: Angle of internal friction, $\phi = 40°$. Natural soil density, $\gamma = 110$ lb./cu. ft. Submerged soil density, $\gamma_s = 60$ lb./cu. ft.

RETAINING WALLS USED FOR COMPARISON OF DESIGN METHODS

It was instructive to compare the results of the Author's method and those obtained from the conventional fixed-support procedure. *Figs 28* showed four walls, A, B, C, and D, with retained heights of 10, 20, 30, and 40 feet respectively. The bending moments induced in those walls were given in *Figs 29* and *30* for loose and dense sands respectively. The bending moments were plotted against a base of log ρ and those derived from the conventional method were represented by horizontal straight lines since they were not, of course, affected by the flexibility of the wall. The moments obtained by the Author's method were derived from sections of

steel piling of the Larssen type and the variation in bending moment with the section of piling was clearly illustrated. It would also be seen that a substantial reduction of moment was obtained by using a lighter section

Fig. 29



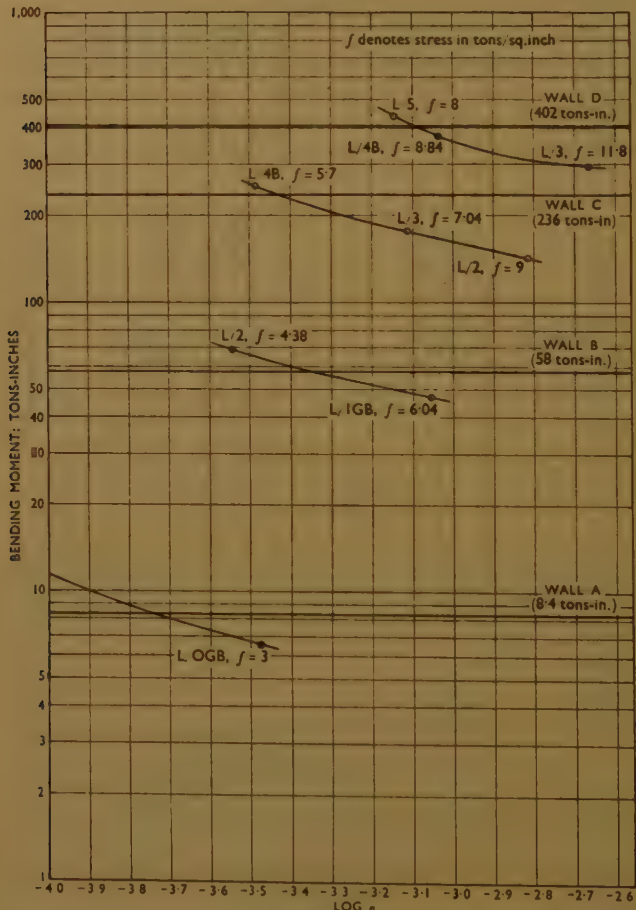
Loose sand, $\phi = 30°$.

Comparison of Author's Method with Conventional Fixed-Arch Support Method

and thus at the expense of only a relatively small increase in stress. Other design data were given in Tables 6 and 7, which also showed the bending moments obtained from the conventional fixed support method in relation to the Author's free support method, which he used as his basis of comparison, that was to say, the 100 per cent moments in *Figs 13* and *14*.

The Figures and Table showed that in many cases the Author's metho gave a larger bending moment than the fixed-support procedure, especiall for the larger walls in loose sand. The explanation seemed to lie in th

Fig. 30



Dense sand, $\phi = 30°$.

COMPARISON OF AUTHOR'S METHOD WITH CONVENTIONAL FIXED-ARCH SUPPORT METHOD

curves of *Figs 16* for loose soils, which show that assumptions of fixity in conventional designs for those conditions were not justified. That was emphasized by the fact that the conventional procedure used in obtaining the data had made no allowance for reduction in bending moment due to arching, though that was commonly done in practice on the basis of

TABLE 6.—LOOSE SAND ($\phi = 30$ DEGREES)

Wall	Steel piling section	Bending moment: tons-inches: Author's free-support bending moment	Bending moment: tons-inches: Conventional fixed-support method	Tie-rod loads per lin. ft. of waling: tons: Author	Tie-rod loads per lin. ft. of waling: tons: Conventional	Pile length and penetration: feet: Author: Length: feet	Author: Pen.: feet	Conventional: Length: feet	Conventional: Pen.: feet
A	0 G B	37	20	0·56	0·59	18·75	8·75	18	8
	1 G B			0·64					
B	1 G B	200	102	1·64	2·12	34·75	14·75	35·5	15·5
	2			1·96					
	3			2·03					
C	4 B	652	378	4·13	4·6	52	22	52·5	22·5
	5			4·4					
D	6	1,560	933	7·68	8·1	63·3	25·3	68·4	28·4
	7			8·1					

Stroyer's research when the wall retained non-cohesive soil and the properties of the soil could be ascertained with sufficient accuracy. However, the Author's earth-pressure tests had demonstrated that, with loose non-cohesive soils, arching and boundary effects were destroyed for stiff walls anchored to yielding supports. Further earth-pressure tests appeared to be necessary to ascertain whether those effects were also apparent with dense soils and behind flexible walls.

In *Fig. 15* (*e*), p. 46, the Author had referred to the reduction in bending moment caused by end thrust. It was easy to visualize the formation of vertical forces which tended to straighten a deflected pile but, since the reactions had to be taken up by the friction of the pile below dredged level and by the bearing of the tie-rods on the ground, it seemed that the reduction in moment could only be very small. For example, a wall of substantial height might have a bending moment of 450 tons-inches and a deflexion of 3 inches. To effect a moment reduction of only 5 per cent, that was, 22·5 tons-inches, a force of 7·5 tons per lineal foot of wall would be necessary. That was equivalent to 60 tons per tie-rod if they were spaced at 8-foot centres. Since the rods would be only a few inches in diameter and too flexible for much of their length to be mobilized to resist bearing pressures, it seemed clear that any moment reduction caused by end thrust would be too small to justify taking it into account.

TABLE 7.—DENSE SAND ($\phi = 40$ DEGREES)

Wall	Steel piling section	Bending moment: tons-inches		Tie-rod loads per lin. ft. of waling: tons		Pile length and penetration: feet			
		Author's free-support bending moment	Conventional fixed-support method	Author	Conventional	Author		Conventional	
						Length: feet	Pen.: feet	Length: feet	Pen.: feet
A	0 G B	14·5	8·4	0·31	0·3	14·5	4·5	15	5
B	1 G B	103	58	1·21	1·15	27	7	29	9
	2			1·39					
C	2	348	236	2·14	2·75	41·1	11·1	43·8	13·8
	3			2·32					
	4 B			2·6					
D	3	760	402	3·72	4·3	53·2	13·2	57·4	17·4
	4 B			4·01					
	5			4·2					

Author's Method.—Pressure diagrams calculated from Coulomb coefficients. Active wall friction $\delta = \frac{2}{3}\phi$, passive $\delta = 0$; Coulomb passive coefficient divided by 1·5.

Shear force at toe $= T_s = \frac{\tan\delta}{1\cdot5}[(P_a - P_p)\tan\delta + w_s H]$. This force together with the passive resistance assumed to act at $\frac{1}{3}D$ from the toe of the pile.

Relation between τ and ρ derived from empirical formula on p. 54 of the Paper.

Conventional Method.—Pressure diagrams calculated from Coulomb coefficients with wall friction $\delta = 20$ degrees (passive only), fixed earth support.

The Author's method was based on first calculating the penetration and free-earth-support moment for a stiff wall from the Coulomb theory as modified by the Author and then obtaining τ_{max}. A "structural curve" showing the relation between the logarithm of the flexibility number ρ of a pile and the value of τ which it could withstand was then drawn and that was superimposed on the "operating curve," which was obtained by interpolation from *Fig. 14*, according to the relative density of the ground. The intersection of the two curves (for example, *Figs 19*) gave the flexibility number corresponding to the actual value of τ and to the specified working stress. Hence the moment of inertia and the dimensions of the pile could

[b]e determined. To draw the structural curve it was necessary to obtain [a] relation between I and y and, although that could be done quite accurately [fo]r timber piles of rectangular section, the wide range of sections in any [t]ype of steel piling might lead to considerable errors. For the particular [c]ase of Larssen sections a reasonably accurate relation was given by the [e]xpression

$$\tau = \frac{0 \cdot 1925}{\sqrt[3]{H\rho^2}}$$

[i]n place of the one on p. 54. However, that was applicable only to sections [w]ith moduli between 15 and 60 inches ^{3} per foot. Outside those limits [o]ther expressions for τ would have to be worked out.

The Author had stated that the use of high-tensile steel permitted the lighter sections to be employed, not only on account of the higher allowable stress but also because of the reduced bending moment which they had to sustain. However, in practice the selection of the most suitable section was also governed by its ability to be driven into the ground without damage and that was frequently the deciding factor.

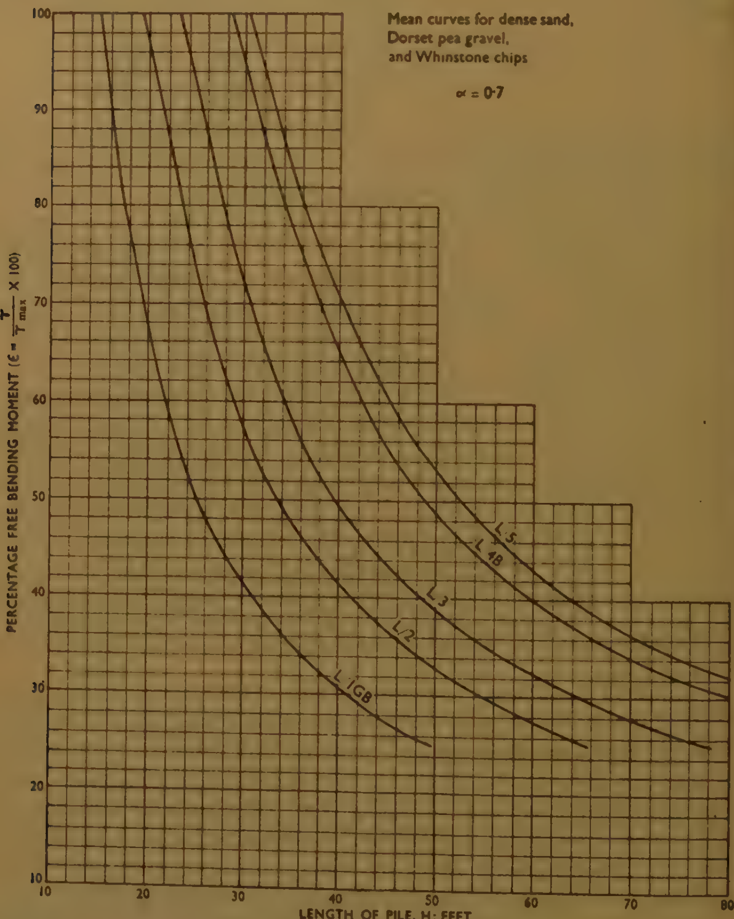
Following on the above review of the Author's method it was possible to suggest a simplified procedure which should make it much easier to apply the results of his research in practice. In *Figs 12* the values of τ_{max} for stiff walls had been calculated from the Coulomb theory as modified by the Author and were indicated by horizontal lines which conformed very closely to the maximum moments measured in the tests. For the purpose of practical design it was possible to avoid the necessity of evaluating the magnitude of T_s. In its place it could be assumed that passive wall-friction developed to the extent of $\delta = \frac{\phi}{4}$, practically the same values of τ_{max} then being obtained. Once the penetration and the maximum value of τ_{max} had been obtained on that basis by any convenient algebraic or graphical method, the value of log ρ could be calculated for the section that had been provisionally selected and the actual moment determined by using *Fig. 14* as a graph of reduction factors. That procedure eliminated the necessity for plotting structural curves and superimposing them on the operating curves derived from *Fig. 14* and therefore seemed much simpler than the Author's.

A disadvantage of the method outlined above was that it did not give such a clear comparison between the stresses (as distinct from bending moments) in different sections as, for example, *Figs 25*. However, that seemed negligible in comparison with the advantages of eliminating the necessity for plotting structural and operating curves, especially because of the inaccuracies which might be inherent in the former.

An even simpler procedure was illustrated by *Figs 31* and *32*. The relation between the pile length H and the percentage of τ_{max} which the wall had to withstand could be represented with sufficient accuracy by a

single curve for each section of piling, relative soil density and value of α. It was then necessary only to determine H and τ_{max} and to obtain the reduction factor from *Fig. 31* or *Fig. 32* for any selected section. Hence

Fig. 31



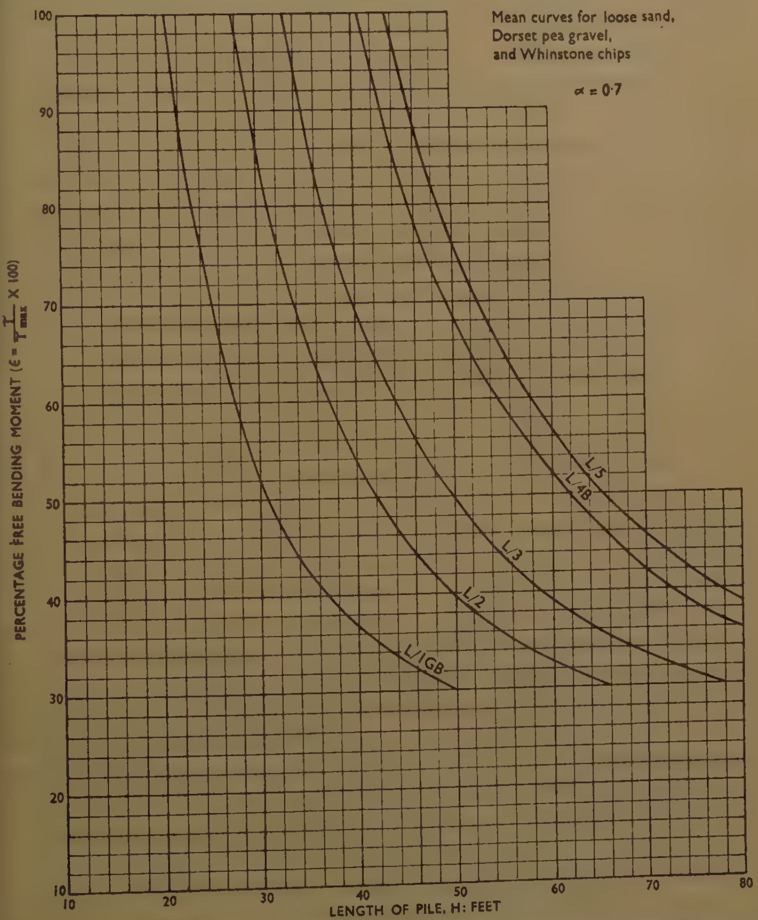
Bending Moment Reduction Factors for Larssen Anchored Steel Pile Walls in Dense Sand with Anchor Yield

the bending moment and, therefore, the stress in the section could be determined.

The Author had shown in *Fig. 22 (a)* that the actual tie-rod loads were smaller than those calculated by taking moments about the centre of pressure of the passive resistance for the free-earth-support diagram. He had also stated that in his tests the load on any one tie-rod could be

[5]0 per cent more than the average, owing to differential yield. It seemed doubtful that such a large increase was likely to be obtained in practice for a wall of substantial height, because the tie-rod spacing was then

Fig. 32



BENDING MOMENT REDUCTION FACTORS FOR LARSSEN ANCHORED STEEL PILE WALLS IN LOOSE SAND WITH ANCHOR YIELD

likely to be between $\frac{H}{10}$ and $\frac{H}{5}$, as compared with about $\frac{H}{3}$ on the model. An allowance of 10 per cent for the effect of differential yield should then be ample but it was agreed that for small retaining walls the Author's suggestion of 25 per cent was reasonable. However, it also appeared that a

distinction should be made between dense and loose soils because with the latter the effect of differential yield could be expected to be very small irrespective of the value of H.

It was agreed that the Author was justified in attributing the reduction in bending moment mainly to the rise in the point of contraflexure caused by the high passive pressures that developed near dredged level when wall friction came into action. He had also stated that there was a secondary source of moment reduction arising from the vertical end thrust caused by the deflexion of the piling but, as shown earlier by Messrs Packshaw and Owen Lake, that reduction was negligible. Nevertheless, some end thrust might exist as a result of vertical arching in the soil, the reaction being provided by wall friction. According to the Author, all arching was destroyed by tie-rod yield but if the bending moments of *Figs 16* were analysed by double differentiation, confirmation of the existence of a secondary cause of moment reduction was obtained. The limitations of double differentiation did not permit definite conclusions to be drawn but there seemed to be evidence either of vertical arching together with limited end thrust or of the influence of the semi-rigid boundary caused by the soil below dredged level. All those effects might, of course, operate simultaneously.

The Author's earth-pressure tests had been confined to a stiff wall (log $\rho = -3{\cdot}32$) and there was therefore no definite evidence about the pressure distribution on flexible walls. It was possible that the distribution might not be in accordance with the Coulomb theory, even with yielding tie-rods. For example, a reduction in pressure might be achieved by transferring some of the pressure from the central area to the tie-rods and to the soil below dredged level by shear when the wall deflected. If the extension of the tie-rods exceeded the maximum deflexion of the wall it was probable that the horizontal shearing forces in the soil were redistributed, but if the extension was less than the deflexion the shearing forces might markedly reduce the intensity of pressure in the central portion of the wall. Doubts might then be expressed about the Author's assumption that there was no difference between a " driven and dredged " and a " backfilled " wall, because the movement between horizontal planes would be greater for the former than for the " backfilled " wall, in which the deflexion increased progressively with the filling, and there was therefore less differential movement between adjacent planes in the backfill. However, perhaps the difference in curvature between the two types of walls was too small to be of any importance.

Attention should also be drawn to the fact that all the soils used by the Author, except the ashes, had been very uniformly graded and would therefore tend to be rather unstable in a loose state. That had been their condition at least down to a depth $0{\cdot}6H$ in all the tests. It would be desirable to know whether the tie-rod yield for walls retaining dense non-cohesive soils would be equally effective in destroying arching and boundary effects.

It was regrettable that the opportunities for checking the Author's theories on full-size walls were so rare, because in practice it was usually impossible to ascertain the properties of soils (and hence their theoretical earth pressures) with a sufficient degree of accuracy. However, several useful lines of investigation on models could be suggested, for instance, double-wall cofferdams, retaining walls in clay soils, pile dolphins, and so on. The Author would render an invaluable service to the engineering profession and to the national economy by continuing his research.

The Author, in reply, thanked Professor Tschebotarioff for pointing out that the active-pressure diagram by the Danish Rules ended at two-thirds of the calculated depth of embedment below the dredge level. The translation of the Rules [24] had been consulted where the distinction between *actual* and *calculated* depths had not been made at all clear. It appeared that the Danish Society therefore made provision for fixity as well as for arching, although that rule varied the degree of arching with flexibility and not the degree of fixity. The findings on p. 33 were not affected by correction with *Figs 27* applied to *Figs 8*.

Professor Tschebotarioff had wondered why the concrete piling designed to moment D had withstood a possible moment given by Curve 1 in *Figs 27*. The reasons were :—

(*a*) Sections designed to moment D would be more flexible and moment 1 would not operate. For instance, the pier at Aalborg (*Fig. 101* in reference 4, p. 70) had a flexibility number given by log $\rho = -2{\cdot}93$ so that less than 60 per cent of moment 1 would operate.

(*b*) Many reinforced-concrete walls designed to the Danish Rules showed cracks. When a reinforced-concrete beam cracked, its effective *EI* decreased rapidly and the flexibility number increased, so reducing the operating moment still further.

Those two reasons had been the *fundamental* cause for the stability of anchored walls in bending, as stated on p. 47. However, there were certain reasons in addition to those given by the Professor.

(*c*) The value of ϕ used for design was probably lower than that in the field.

(*d*) Yield and cracking might have induced a state of arching in walls anchored to elastic supports, since no further yield would occur immediately (see also p. 647).

The driving of the sheeting into a loose sand would compact the subsoil locally but would not alter the compressibility and the shear stress/strain relationship for the bulk of the soil on the passive side, so that the possibility of the existence of loose sand could not be dismissed.

Referring to Stroyer's moment reduction factor, it had been stated on

[24] See reference 1, p. 70.

p. 36, item 6, that agreement was obtained with Stroyer's work for un- yielding supports because the model behaved as if pinned at the anch and at a distance from the toe equal to one-third of the penetration dept —that was to say, as a stiff pile. That was nothing whatever to do wit the results of *Figs 16*, which applied to *yielding walls with no arching actio* and where the walls in any case did not all behave as if pinned, as i Stroyer's tests. To avoid any implication of a general numerical agree- ment with Stroyer for yielding walls the Author had made the statement given in the centre of p. 47.

Professor Tschebotarioff had denied that his tests had been limited t fixity conditions. Whilst it was certain that the Professor had not se out purposely to study fixity conditions alone, that was in effect wha had happened. Since the flexibility range he had used governed his fina conclusions it was instructive to study that matter a little more fully.

In *Fig. 22* of his Report,[25] Professor Tschebotarioff had given th general lay-out of his twelve tests with sand backfills, and the Author now added a list of those tests, including Test 57 (*Fig. 23* of the Report) to- gether with the log ρ values of the models.

Test	51, 51–A, 52	55, 55–A	53, 57, 58–IV	54, 54A, 59	56, 56–A
log ρ	$-2{\cdot}01$	$-2{\cdot}31$	$-2{\cdot}54$	$-2{\cdot}74$	$-2{\cdot}96$

All but two of those models had a log ρ value equal to or greater than $-2{\cdot}74$. Those corresponded to "fixity condition" as understood in the past, and the models with log $\rho = -2{\cdot}96$ would also have remained fixed in dense subsoil as in Test 56 and would never have been Freely Supported even in loose subsoil, Test 56–A.

On p. 5 of his Report, Professor Tschebotarioff had outlined his method of arriving at the model flexibility, which was to produce a model which would behave similarly to a prototype. He had used a "model shape factor F" where :

$$F = \left[\frac{CP}{CM}\right] \times \frac{1}{n}$$

and CP denoted distance from the neutral axis to the extreme fibre of a *prototype* sheet-pile

CM denoted same value on the model $= \frac{1}{2}$ inch

and n denoted model scale.

The Professor had then gone on to state "Taking $CP = 5{\cdot}75$ inches and $n = 5$ we obtain $F = 2{\cdot}30$." Why were those values used ? The Author could only assume that the model was intended to simulate an average type of field pile already built. A similar procedure was adopted for the third

[25] See reference 4, p. 70.

eries of tests (p. 36 of the Report) " to produce close similarity of bulkhead deflexion on the model with those on the *prototype*." If the field piles chosen as prototypes had been originally designed assuming fixity and possibly some form of arching reduction, then those piles would have been flexible with log ρ values of -3 to -2.5 and the model values would have been chosen over the same range as shown in the Table. Professor Tschebotarioff, in defence of his statement, had quoted Test 57, stage F, at $\alpha = 0.89$. That had been the last stage in the test, just prior to passive failure, $\alpha = 0.89$, and where Free Earth Condition would operate in any case at failure. In that test (*Fig. 93* of the Report) the point of zero moment at stage F had been well below the dredge level, as also in *Figs 88* and *89* for Tests 56 A. Professor Tschebotarioff had ignored those results in his final proposals, using only the results which applied to models simulating the " fixed " prototype, otherwise he could not have concluded that all piling should be designed as if pinned at the dredge level. The Author felt justified therefore in his summary on p. 29 that Professor Tschebotarioff's work had been limited to fixed-earth-support conditions.

Professor Tschebotarioff still believed that the work of Browzin [26] was in error owing to the rough method of measurement. Whilst the accuracy of his measurements bore no comparison to the strain gauges used by the Professor, the contradiction did not lie there. Browzin had made errors in calculating the position of field piles on his *Fig. 16* and had neglected the shape factor. Further errors had occurred by representing the stiffness of his models by the slenderness ratio, which was not a universal constant. Making all necessary corrections, Professor Tschebotarioff's results would be found to lie in the region of Browzin's Type I deflexion along with field piles designed to " fixity," whilst piling designed to free-earth-support conditions, and the Author's original large model, lay in the region of Type II. Thus Browzin's conclusions were in error and his arbitrary division line between the two types of deflexion had no definite significance, but his model observations were nevertheless in agreement with all other known work.

The Author agreed that " Fixed earth support of bulkheads frequently was a reality . . ." but the references recommended by Professor Tschebotarioff for sheet-pile wall design gave no guidance as to the degree of the fixity which would occur with varying types of wall, subsoil, and backfill conditions.

The Author was very grateful to Professor Karl Terzaghi for his generous opening remarks and for his valuable criticisms.

Professor Terzaghi was correct in stating that the agreement with the Coulomb theory with no wall friction in the initial tests, on just filling the bin, was merely coincidence. The statement had been made to indicate in summary form that the pressure cells recorded a triangular

[26] See reference 5, p. 70.

distribution at the start, of that order. Small disturbance caused by th release of the hanging supports and the difficulty in setting the tie-ro "just tight" without applying passive pressure behind the ties would ha prevented the full "at rest" active pressure from acting at that stag Some tests had shown that a line representing a K_A-value 10 per cen higher than the "no wall-friction value" could be drawn through th points. At that stage the wall did not bend, so that no strain-gaug readings were available as a check. No comparison could be made be tween Professor Terzaghi's highly accurate measurements on the large model retaining wall ever constructed, and the accuracy of the Author pressure-cell readings which were necessarily scattered. The gauges wer intended solely to illustrate that the distribution of active pressure wa triangular to commence with, changing to definite arching on dredging and reverting to a triangular distribution on yield.

The data in Table 5 related to a sand having an ultimate value o $\phi = 34$ degrees, and at $H/1000$ yield at the top, the K_A value on the sti wall was 17 per cent higher than the $\delta = \frac{2}{3}\phi$ value. The initial model-pili results at $\alpha = 0{\cdot}7$ were much more accurate than at the start since th strain gauges recorded the bending moment diagram to a high degree o accuracy. The pressure diagram had to have the shape indicated by th pressure cells and also had to agree with the bending moment diagram and it was not likely that an error of 17 per cent or anything like it could occur. Further, the maximum moments for stiff piling in loose soil in th flexibility tests never exceeded the $\delta = \frac{2}{3}\phi$ value. The Author suggeste that Professor Terzaghi's figures relating the mobilization of ϕ with wal outward movement could only apply to an infinitely stiff wall, such as he had used. The deflexions at the centre of the piling were of the order of $H/200$ and at that value the K_A values of Table 5 were within 7 per cent of the $\delta = \frac{2}{3}\phi$ value, which was approaching the overall limits of accuracy of the model pile tests. Wall flexure therefore, mobilized ϕ more rapidly. The side vibrations would have given a larger moment rather than a smaller one if they had affected δ and ϕ. Instead they were of such small amplitude that the soil adjacent to the model wall was not disturbed.

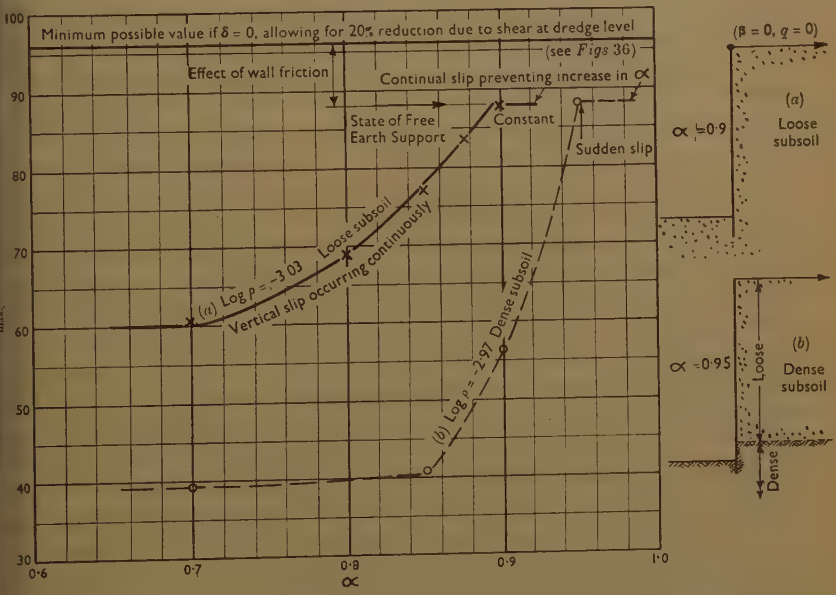
The effect of scale on the soil compressibility had been stated near the foot of p. 51, but the Author agreed that it might have been better to have included it on p. 37.

Professor Terzaghi's analysis of the factors affecting the toe shear force was important. It was perfectly true that a small vertical movement eliminated wall friction, but it was possible that the extra lateral load resulting from that induced further bending which mobilized wall friction *very* rapidly, after the manner shown in Table 5. Also vertical slips only commenced when the penetration depth was sufficiently small to allow small passive slips to occur, which also mobilized wall friction. In order to test what the outcome would be, the Author had conducted tests with loose and dense subsoil and loose backfill, recording the maximum bending

moment with dredging. The maximum free-earth-support value of the bending moment τ_m, taking $\delta = \frac{2}{3}\phi$, had been calculated for each value of the dredge level, given by α, and the observed percentage maximum moments were shown in *Figs 33*, as a percentage of τ_m, plotted against α. With loose subsoil the percentage moment had increased with dredging to 88 per cent at $\alpha = 0{\cdot}9$. At that value the vertical slips had been large and had kept pace with the dredging so that α and ϵ remained constant. The pile had failed in bearing and not in passive failure.

However, taking $\delta = 0$ and allowing for a reduction of 20 per cent caused

Figs 33



EXPERIMENTAL EVIDENCE THAT WALL FRICTION IS MAINTAINED ON A FLEXIBLE WALL EVEN THOUGH VERTICAL SLIP OCCURS

by shear at the dredge level (a generous figure, since only 10 per cent would be likely to occur at $\log \rho = -3{\cdot}0$, *Fig. 36* (*a*), to be discussed later) the percentage moment ϵ should have been at least 96 per cent. Exactly the same occurred with dense subsoil, except that in that case the passive failure and vertical slip occurred simultaneously at $\alpha = 0{\cdot}95$. Thus, on balance, wall friction would certainly have been mobilized and the vertical thrust on the toe maintained. The wall had moved down consistently from the start of vertical slip, illustrating that the vertical thrust had been maintained.

The value of $\alpha = 0{\cdot}95$ for passive failure meant that if toe shear was neglected a value of $K_p = 67$ would have to have acted just prior to failure.

That value was quite absurd and there could be no doubt that toe shear occurred in the model. However, owing to effects of soil compressibility upon the bearing capacity of deep foundations, it was likely that the behaviour of model piles in bearing could not be directly scaled up to that of a steel field pile. It was true that the bearing capacity of loose sand was negligible, but the equations on p. 65 also gave a negligible value for T_s when ϕ was low. The point in question was the limit of the bearing capacity of field piles driven into dense soil, since it was there that the equation on p. 65 led to a small saving in penetration depth. For concrete piling having a toe bearing width of the order of 12 to 18 inches, the ratios of penetration-depth to base-width lay in the range 6–20 and at those values the results for deep foundations as given by G. G. Meyerhof[27] could possibly be applied. Those results showed that the bearing capacity was always greater than the vertical pressure applied at the toe. For example, the required bearing capacity in *Figs 24* for the concrete section was not greater than 4 tons per square foot and that would be likely to be sustained by dense subsoil.

For steel piling having a thickness generally not exceeding ½ inch, very high bearing capacity values were required to maintain a toe shear force. For instance, the equivalent steel sheet-pile construction in *Figs 24* required a bearing capacity of more than 100 tons per square foot, which was quite fantastic, and although very high bearing pressures resulted from extrapolation of results of tests on a deep foundation to the case of a sheet pile having a depth/width ratio in the range 250–400, that was a matter of considerable uncertainty owing to the increased effect of compressibility at those high ratios. Whilst the Author could not agree that the vertical thrust on sheet-piling in general was negligible, purely from the direct observations in the experiments, Professor Terzaghi was most certainly correct in pointing out that the toe shear force should be neglected for steel piling unless driven on to rock or hardpan. The Author thought that the toe shear force could be relied upon for concrete piling. However, it was not worth introducing into the calculations unless the piling was driven into fairly dense subsoil.

The Author appreciated the fact that there were many sites where the soil varied considerably in density, but it was difficult to see how the proposed charts could be seriously misapplied, provided the subsoil was at least known to be sand. For flexibility numbers greater than $\log \rho = -3{\cdot}0$, in the region normally used, difference in moment between extreme looseness and extreme denseness was not more than 10 per cent of the free-earth value, that was to say, 20 per cent of the values being used. Thus if a mean value of relative density given by $D_r = 0{\cdot}5$ was used, serious errors could not arise. Care was necessary to ascertain the possible occurrence of pockets of silt or clay having a higher compressibility than loose sand.

[27] "The ultimate bearing capacity of foundations." *Géotechnique*, vol. II, p. 301 (Dec. 1951).

Where the conditions of the subsoil were completely unknown or contained random large pockets of highly compressible material no moment reduction from free-earth support conditions should be allowed. The Author agreed that no charts of any kind should be applied to any soil mechanics problems without the overriding judgement of the engineer. The Author would have thought that the use of the proposed charts as the background for design and modified as necessary by soil conditions and the experience of the engineer was the right answer.

It was agreed that the values of EI of sheet piles with frictionless locks, and when locked together and subject to the Poisson-ratio effect, would be useful and might be supplied by the makers.

The "equations" at the foot of p. 60 were not equations of the left-hand side to the right-hand side since the proportional sign was used. Thus it was not necessary that the units should agree so long as they were not changed.

For example, the deflexion of a simply supported beam subject to triangular loading was given by:

$$d = 0{\cdot}01304 \frac{K\gamma(\xi H)^2}{2} \cdot \frac{(\xi H)^3}{EI} \text{ feet}$$

where

$$\frac{K\gamma(\xi H)^2}{2} = \text{loading in lb. per foot}$$

$$\xi H = \text{beam-length in feet}$$

$$EI = \text{stiffness in lb. per square foot per foot}$$

If the value of (EI) in lb. per square inch per foot was inserted one could write:

$$d = 0{\cdot}01304 \times \frac{144K\gamma}{2} \cdot \frac{(\xi H)^5}{(EI)}$$

$$\text{or } \frac{d}{H} \propto \xi^5\rho$$

This "equation" illustrated, whatever the units, that deflexions increased with flexibility ρ but rapidly decreased with small decrease in ξ, which in turn decreased with increase in ρ. The exact values of $\frac{d}{H}$ were not derived, since it was sufficient to prove that deflexions did not increase rapidly with increase in pile flexibility, until of course full fixity was obtained at very high flexibilities beyond the practical design range. The values of $\frac{d}{H} = 0{\cdot}004$ to $0{\cdot}006$, on p. 62, had been obtained from a wide range of test results.

Mr Savile Packshaw and Mr J. Owen Lake had made a valuable contribution to the discussion which would be instructive to read with the Paper. The Author was particularly interested in *Figs 28–30* which

demonstrated clearly some of his remarks in Table 4, namely, that large structures in loose soil with high surcharge and differential water pressure required conservative design compared to the Fixed Earth Support Analysis, but that economies were obtained with dense subsoil and especially when no surcharge occurred and with smaller structures.

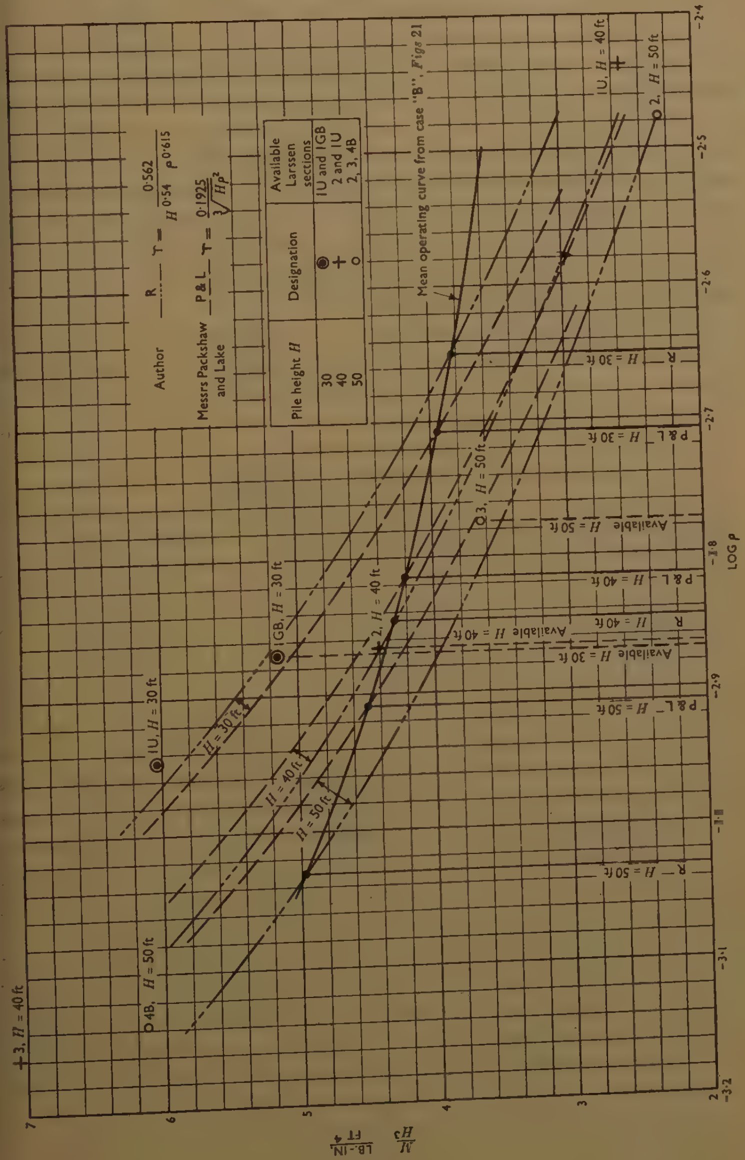
It had been implied that the formula for Larssen sections relating τ and ρ on p. 54 was not very accurate and that the use of structural curves for steel piling "might lead to considerable errors." The Author was unable to agree with that. A large-scale diagram of the moment/flexibility diagram in the practical range $\log \rho = -3{\cdot}1$ to $-2{\cdot}5$ was shown in *Fig. 34* and the structural curves for H=30, 40, and 50 feet were plotted according to the relations given by Mr Packshaw and Mr Lake and the Author. On the diagram were plotted the actual flexibility numbers of the available sections. A mean operating curve likely in practice, curve B in *Figs 21,* was plotted and it was instructive to consider the effect of the alternative relations on the ultimate design. Table 8 showed the chosen sections and estimated stresses.

TABLE 8

Pile height, H: feet	Theoretical design value of $\log \rho$		Available section and $\log \rho$	Final stress		
	Messrs Packshaw and Lake	Author		Actual	Messrs Packshaw and Lake	Author
50	−2·910	−3·030	−2·775	9·1	8·95	9·45
40	−2·815	−2·845	−2·865	7·88	7·65	7·92
30	−2·707	−2·650	−2·870	6·75	6·82	6·66

With regard to the alternative formula, it gave a closer estimate to the actual stress by 0·2 ton per square inch for the 50-foot pile, but was further out than the Author's estimate in the 40-foot pile by 0·19 ton per square inch, whilst both values were equally near for the 30-foot height. Thus there was little in it either way. A glance at *Fig. 34* was sufficient to show that either relation led to the same pile section being chosen. The suggested trial-and-error method seemed laborious in view of the fact that the final sections could be chosen so rapidly from the structural curve and the actual stress could always be finally checked. Instead of plotting the approximate τ/ρ relation one could plot the actual section positions as shown in *Fig. 34*. They could be plotted once and for all time and the operating curve superimposed on tracing paper.

The use of *Figs 31–32* was a sound and simple method which could be useful to a designer working with one type of piling and one type of section. However, the civil engineer who wished to decide whether to



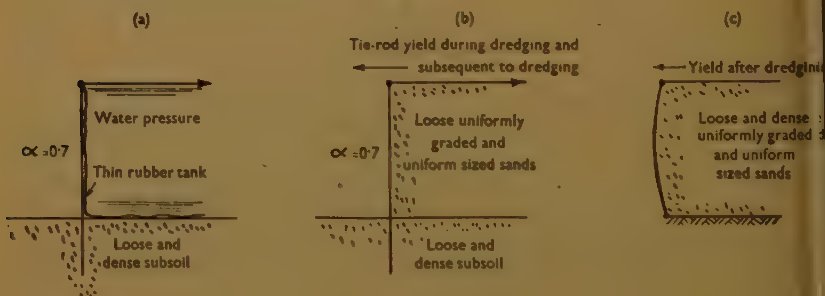
Comparison of the Author's Formula for Larssen Steel Sections and that Proposed by Mr Packshaw and Mr Lake

build his wall in wood, concrete, or steel, and which type of steel section was desirable, could make his decisions quicker and more easily by the use of the structural and operating curves than by comparing a large number of different curves, as in *Figs 31–32*, to cover all possible types of pile. It would be useful, however, if makers could include, in the literature on their sections, the values of τ and $\log \rho$ for each available section over a series of pile heights.

Mr Packshaw and Mr Lake had raised important questions regarding the sources of secondary moment reduction. It was true that the Author had been unable to repeat his pressure-cell tests on a large flexible wall. Agreement in results had been obtained with Professor Tschebotarioff who had shown that moment reduction arose from shear stresses set up between the backfill and the subsoil for flexible walls.

In order to investigate the variation in the degree of reduction caused by shear stress at the dredge level with the flexibility of pile, and also the

Figs 35



influence of arching under the conditions questioned by Mr Packshaw and Mr Lake, additional experiments as shown in *Figs. 35* had been made. Tests had been made with a water backfill (*Fig. 35* (*a*)) so that there could be no possible pressure reduction above the dredge level. In those tests, moment reduction with yield of $H/1{,}000$, arising from cantilever action, occurred only to a small degree with very stiff sections in dense soil, so that the effect could be ignored. Further tests were also made with soil backfill (*Fig. 35* (*b*)) using both uniformly graded and uniformly sized particles in two separate series of tests, for the backfilled and driven-and-dredged conditions, and for the cases of no yield, yield during dredging using springs for tie-rods, and yield subsequent to dredging. All those tests had been made for the conditions of loose and dense subsoil. Finally a highly flexible model-wall (*Fig. 35* (*c*)) 3 feet long and 1 foot high, pinned at both ends, had been built in brass sheet 0·038 inch thick; it had carried strain gauges down the centre. The flexibility number, taking the height of 1 foot as H, was given by $\log \rho = -2{\cdot}95$. However, if the model was assumed to represent part of an actual pile which behaved as if pinned at

the top and at a point near the dredge level, then the equivalent height of the field pile would be about 1/0·7 (=1·43) foot and the equivalent flexibility number was given by log $\rho = -2{\cdot}33$. The top pin had been capable of controlled release. The pinned wall had been calibrated directly by water pressure so that the Poisson-ratio effect was eliminated. The case of $\beta = 0$, $q = 0$ was taken for the model-pile experiments in order to facilitate comparison with the pinned-wall tests.

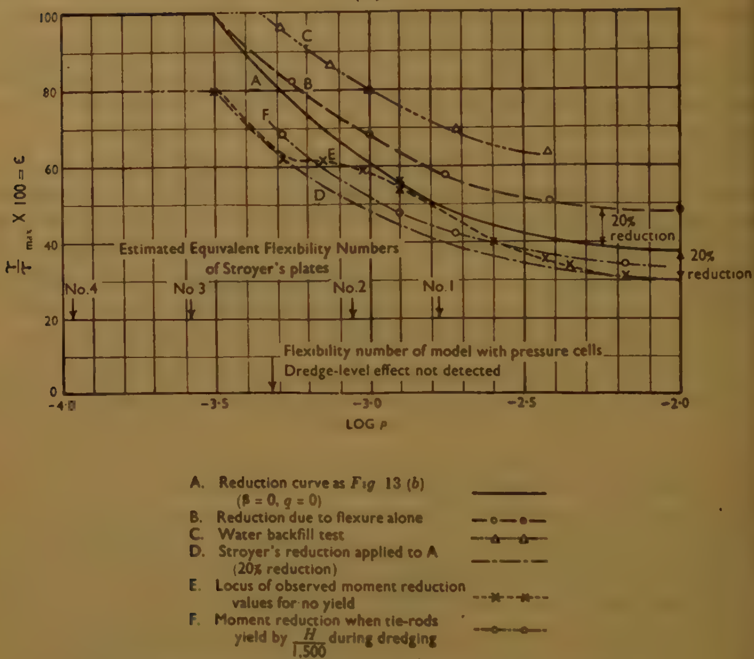
The results of the model-pile tests were shown in *Figs 36*. Curve A showed the percentage moment reduction on a backfilled pile subject to a negligibly small yield or a driven-and-dredged pile subject to a subsequent yield of the order of $H/1000$. The curve corresponded to *Fig. 13* (*b*) for $\beta = 0$, $q = 0$ within 1 per cent, although the apparatus used had been completely new. The locus of the moments obtained by taking moments about the point of zero moment assuming a triangular distribution above the dredge level was given in curve B. The results for the water tests were shown in curve C, and for that case the moments obtained by taking moments about the point of zero moment agreed with the maximum values observed. It was seen that the shape of curve C was similar to curve B but that the Critical Flexibility Number was displaced to the right. That was because the lateral load caused by the water pressure was about 2½ times that caused by the soil backing, so that the passive soil was compressed further with the extra load. That was equivalent to a pile with soil backing, but driven into a subsoil of greater compressibility, although not as great as 2½ times the compressibility of loose sand, since the stress/strain curve was not linear with pressure. Thus, curves B and C were in agreement and also provided completely independent confirmation of the Author's conclusions with regard to the influence of soil compressibility on the critical flexibility—an influence of the greatest importance with regard to the stress on the sheeting.

The difference between curves A and B arose therefore from a secondary source of reduotion and not due to flexure. It could not have been caused by vertical arching because very large tie-rod yields had been permitted in an effort to increase the moment, without success. The difference between A and B became negligible towards the flexibility number used by the Author in his pressure cell tests and that explained why that source of reduction was not detected by the pressure cells. At higher flexibilities the difference increased to 20 per cent of the value due to flexure alone.

Complementary tests with the pinned pile had been made. The pile had first been backfilled with loose sand and the bending-moment distribution observed during tie-rod yield, curve 2, *Fig. 37*. A very small decrease in moment had taken place with large yields. The moment distribution which should have occurred with a triangular pressure distribution was shown in curve 1. Error could not have occurred in the value of K_A for the soil, since the two curves were in agreement at the top of the pile. The moment reduction was 12–14 per cent. The same result

Figs 36

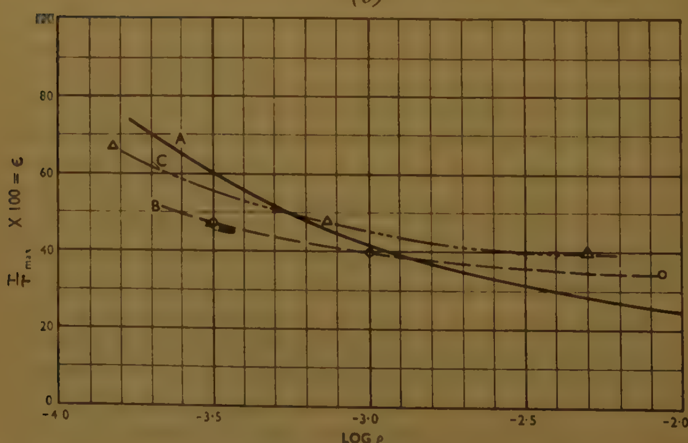
(a)



LOOSE SANDS AND GRAVELS OF VARIOUS GRADINGS

($\beta = 0$, $q = 0$, $\alpha = 0{\cdot}7$)

(b)

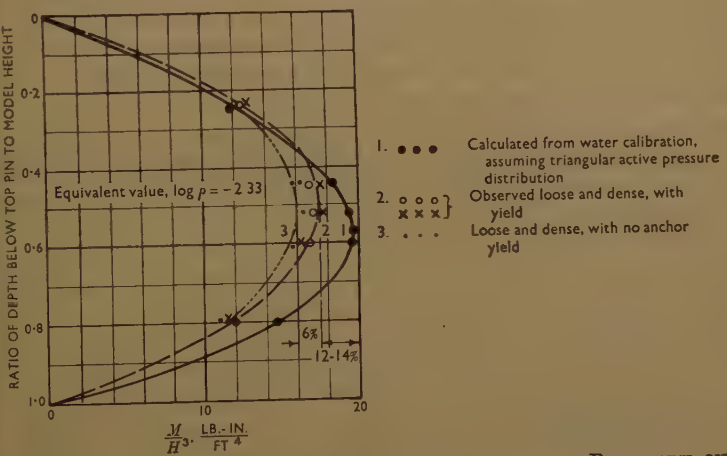


DENSE SAND SUBSOIL

as obtained with Dorset Pea Gravel and a uniformly graded sand and ravel. The reduction was thus 6–8 per cent less than that shown on the ctual model pile, but it should be remembered that the lower fictitious in of the actual pile had moved outwards with deflexion, so mobilizing dditional shear stress in the soil. Thus the difference between A and B rose from the boundary effect at the dredge level and was not caused by rching.

Fig. 36 (*b*) showed similar results with dense subsoil. There the dif- erence between curves B and C was much smaller, since the change in ompression of the dense sand with extra load was not very great. The econdary moment reduction reduced to zero at log $\rho = -2{\cdot}85$, partly

Fig. 37



Moment Reduction Due to Shear Stress at the Lower Boundary on a Pinned Model Pile

because of a reduction in shear stress at the dredge level with less rotation of the pile and partly because of lack of mobilization of ϕ. The observed moments became greater than even curve C with stiff sections, owing to the fact that ϕ was not fully mobilized. Yet tie-rod release of the order of $H/1{,}000$ had occurred. Thus Professor Terzaghi's remarks regarding the mobilization of ϕ were confirmed and the effect of the wall flexure in mobilizing ϕ more rapidly as suggested by the Author was also substantiated.

It was then necessary to study the phenomena of arching. In all the following tests, the model-pile had been prevented from deflexion until it had been backfilled, and deflexion had then been induced both before and during tie-rod yield. Stroyer's moment-reduction factors, taking into account that they applied to the case of $\delta = 0$, had been used to plot curve D. The series of tests made, allowing no tie-rod yield at all with

various flexibilities, gave results which lay on curve E. Thus with stiff walls equivalent to the Author's original model, full agreement occurred between E and D. With increase in flexibility the Author had been unable to obtain any arching at all, with no tie-rod yield, even using the more stable uniformly graded sand and gravel soil backing. With increasing flexibility the arching reduction had been built up again, but by the time full reduction occurred, the value had been extremely sensitive to tie-rod yield. A yield of less than $H/5{,}000$ had been sufficient for the complete destruction of the arching at a flexibility number given by $\log \rho = -2{\cdot}16$

In support of those tests the pinned pile had been backfilled with loose and dense sand, also sand and gravel of both gradings. Wall deflexion had been allowed after filling with no tie-rod yield. The moment distribution with loose sand of both gradings, curve 3, *Fig. 37*, was within 6 per cent of curve 2; that was to say, there was little arching. The result with the dense sand was exactly the same. The reason was that the wall deflexion had been sufficiently large to induce complete slip of the backfill which not only destroyed arching but also reduced the dense sand to the loose state. For the uniformly graded sand and gravel the deflexion had not changed the dense state completely to the loose state and *in that sense* the uniformly graded material was more stable. It was not more stable in arching, however. Owing to the effect of the outward movement of an actual pile at the dredge level the volume increase behind the wall was greater than for the pinned pile, which would therefore be expected to behave similarly to an actual pile of smaller flexibility. Thus the results of the two models were complementary. A small outward deflexion would therefore cause arching to occur at a constant reduction. With increased flexure a point was reached at which the material suddenly slid into the space left by the outward deflexion, even with no yield. Continued deflexion rebuilt arching until a further slide occurred. Just prior to a slide the arching was extremely sensitive to tie-rod yield.

That phenomenon of slides had been observed by Stroyer. The equivalent flexibility numbers of his plates had been estimated and shown on *Fig. 36* (*a*), and it should be considered that, owing to the lack of outward movement of the lower pin, the positions of the plates on the diagram should be displaced to the left by at least $\log \rho = 0{\cdot}3$. Thus only plates 1 and 2 represented, in that respect, flexures near those used in practice. For those plates Stoyer had experienced slides of the material [28] accompanied by a marked rise in pressure which he had attributed to a temporary breakdown in arching.

In an actual field pile, outward yield of the tie-rods occurred during the deflexion of the pile. Since the last amount of dredging would cause a large increase in bending deflexion compared to the extra yield of the tie-rods, some arching would remain. Curve F, *Fig. 36* (*a*), showed the

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[28] See reference 2 (p. 70)—*Fig. 12* and p. 117.

results obtained using springs for the anchorage so that a yield of approximately $H/1{,}500$ occurred during dredging. It was seen that 15 per cent arching reduction remained. There were, however, many reasons why that could not be recommended as reliable on an actual structure :—

(1) The designer would have to guarantee that no subsequent yield of the order of $\frac{1}{4}$ inch on the anchorage occurred during the lifetime of the structure.

(2) It was doubtful whether, even after an exhaustive study of the arching phenomenon in the laboratory and extensive measurements of the behaviour of anchor plates in the field, over a long period of time, one could ever be certain of the degree of stability of the arching in view of the cyclic nature of the influence of the flexure of the pile and the tie-rod yield on the slides.

(3) Sudden outward deflexion caused by yield of the steel or cracking of the concrete wall might not necessarily induce further arching ; it might induce a slide.

The reduction curves given in the Paper (*Fig. 14*) represented absolutely stable values and were nothing whatever to do with the variable phenomena of arching.

The following was a list of the factors affecting moment decrease on flexible walls in order of magnitude and reliability :—

Influence	*Reduction on τ_{max}*
(1) Flexure and soil compressibility	0–70% } 0–76%.
(2) Shear at dredge level	20% of (1) }
(3) Arching	Not absolutely reliable 15% of (1) + (2).
(4) End thrust	Max. value 3%, negligible.
(5) Variation in mobilization of ϕ: included in data with (1) and (2), shown in *Fig. 14*.	

The Author was indebted to Mr Packshaw and Mr Lake for their encouraging closing remarks and suggestions for future research. Professor J. A. L. Matheson had kindly provided the Author with every facility for continued research at Manchester University.

CORRESPONDENCE
on a Paper published in Proceedings Part I, July 1952

Paper No. 5856

"Festival of Britain, 1951: The South Bank Exhibition Buildings"†

by

Sir Hugh Casson, M.A., F.R.I.B.A., Ralph Freeman, C.B.E., M.A., M.I.C.E., and Robert Trafford James, O.B.E., M.I.C.E.

Correspondence

Mr A. M. Muir Wood considered that the most commendable aspect of the Paper lay in its record of the co-operation between architect and engineer, which was essential for the success of such a project. It was to be hoped that the Paper represented the first of a regular series of joint contributions to the Institution from architects and engineers, promoting the better appreciation by each of the other's point of view.

The constituent parts of the South Bank Exhibition displayed widely varying extents of indirect and direct dependence upon the engineer for conception and design. A feature they possessed in common lay in the singleness of purpose shared by structure and architectural treatment; the Dome of Discovery was the one exception, for its essential levity conflicted with the ponderous concrete-work by which it was partly occluded.

The source of the now-receding rivalry between architect and engineer was traceable to the nineteenth-century insistence upon building in the grand or statuesque manner; it had been found that ever more massive or extensive classical porticos or gothic façades could be draped upon a concealed structural skeleton. It was scarcely surprising that, in those circumstances, the engineer had been expected to remain as unobtrusive as his load-bearing structure. The architect, on the other hand, had been free to explore meanwhile his patron's susceptibility to the blandishments of publicity.

The daily press and most other agents of popular instruction appeared to remain largely unaware of the changed relationship between architect and engineer, who were engaged in the return to a style of building that related design to purpose, site conditions and locale, structural methods,

† Proc. Instn Civ. Engrs, Part I, vol. 1, p. 346 (July 1952).

and materials of construction. So long as the greater part of the limited amount of non-domestic building at present permitted by licence was commissioned according to different precepts, the scant regard paid to the engineer's contribution to the lesser part remained regrettable but explicable.

The Authors, in reply, thanked Mr Muir Wood for his remarks, with which they were in complete agreement. They, too, hoped that the Paper would encourage further joint contibutions to the Proceedings of the Institution from engineers and architects, to reinforce the interdependence of each profession upon the other in the successful design and construction of building works.

OBITUARY

MERVYN FREDERICK RYAN, C.B.E., who died at Buenos Aires on April 28th, 1952, at the age of 68 was born at Valetta, Malta, on December 22nd, 1883.

He was educated at Stonyhurst College and University College, Nottingham.

In 1902, he became a pupil at the Derby Locomotive Works of the Midland Railway. In 1906 he went to the United States of America where he continued his practical training, first in the Testing Department of the General Electric Company at Schenectady and later in the Motive Power Department of the Pennsylvania Railroad.

On his return to England he re-joined the Midland Railway as Piece-work Inspector and was subsequently appointed Assistant to the Works Manager. From 1911 to 1913 he was Resident Locomotive Superintendent of the Somerset and Dorset Joint Railway and in 1913 he became Assistant Chief Locomotive Superintendent of the London and South Western Railway. Soon after the outbreak of the 1914-1918 war he was transferred to the Ministry of Munitions and shortly afterwards succeeded Sir Henry Fowler as Director of Munitions Gauges; his services in this capacity were recognized in 1918 by his appointment as a Commander of the Order of the British Empire.

After the war he returned for a short period to the London and South Western Railway as Deputy Chief Mechanical Engineer. In 1919 he went to the Argentine to begin the most important phase of his professional career, first as Chief Mechanical Engineer of the Central Argentine Railway and then successively Assistant General Manager (1925), General Manager (1928), and Managing Director (1945) of the Buenos Aires and Pacific Railway. After his retirement in 1947 he was, for a short time, responsible to the Argentine Government for the management of all the previously British-owned railways. In 1949 he went to India, and in 1950 to Siam as the railway member of the missions sent to those countries by the International Bank for Reconstruction and Development.

He was elected an Associate Member in 1910 and was transferred to the class of Member in 1922. He was also a Member of the Institution of Electrical Engineers, the Institute of Transport and the Institution of Locomotive Engineers, whose President he was in 1918. He took a very keen interest in the development of the Argentine branches of these institutions and played a large part in the formation of the joint centre in Buenos Aires.

He is survived by his wife and two sons of a former marriage.

GEORGE WATSON, who died at South Croydon, Surrey, on the 4th June, 1952, at the age of 83, was born on the 11th November, 1868, at Shirburn, Oxon.

After obtaining his technical education at the University of Leeds, he served a period of apprenticeship with the Leeds firm of John Fowler and Co. Ltd. To further his practical knowledge he became in 1890 a departmental manager with T. R. Harding and Son, Leeds, and in 1891 Assistant Engineer with the Farnley Iron Co. Ltd, Leeds, with whom he remained for a year.

In 1892 he joined the staff of the Horsfall Destructor Co. Ltd, of Pershore (later renamed the New Destructor Co. Ltd), being made Managing Director and Chief Engineer in 1897.

During the years following he was responsible for the design and construction of one hundred and twenty refuse-disposal plants in all parts of Great Britain, and also in Europe, Australia, and the Far East.

In the first World War Mr Watson was for a time engaged in the design and production of a small-type incinerator used in quantity by the army, later he was given the charge of a small munitions works.

In the two years that followed the end of the war in 1918, he was employed by the War Office in the task of valuing and supervising the dismantling of many munitions works throughout the country.

In 1920 he began practising as a consulting engineer, specializing in the design and installation of industrial and municipal refuse-disposal plant and steam-raising equipment.

Mr Watson was elected an Associate Member of the Institution in 1894 and was transferred to the class of Member in 1906.

In 1899 he was awarded a George Stephenson Medal and Telford Medal for his Paper on "Refuse Furnaces." *

He was also a Member of the Institution of Mechanical Engineers.

He leaves a wife and three daughters; his only son pre-deceased him in 1950.

HERBERT HAMER, who died on the 20th March, 1952, at the age of 65, was born on the 10th March, 1887.

He was educated at Accrington Grammar School, Lancashire, and was articled to the late W. J. Newton, Borough Engineer and Surveyor of Accrington, from 1904 to 1908.

On the completion of his pupilage he remained with the Accrington Corporation, first as Assistant Engineer, and finally, from 1912 until 1913, as Deputy Engineer and Surveyor.

In 1913 he was appointed Deputy Borough Engineer and Surveyor of Stockport, a post he held until his enlistment in the Army in 1915. Joining the Cheshire Regiment as a private he was soon transferred to the Corps of

* Min. Proc. Instn Civ. Engrs, vol. 135 (1889–90, Pt. I) p. 300.

Royal Engineers, and he eventually retired from the Army with the rank of Major.

He then spent a year on the staff of the Ministry of Transport as Secretary to the Committee dealing with the taxation of motor vehicles.

In 1920 he returned to Stockport, this time however as Borough Engineer and Surveyor, where he remained for ten years until his appointment in 1930 to be the City Engineer of Hull.

In this capacity he was responsible for the inauguration of many important development schemes, in particular the building of seven housing estates, and a new main drainage system.

In 1937 he was appointed City Engineer and Surveyor of Liverpool, but retired in March 1947 owing to illness (coronary thrombosis).

Mr Hamer was elected an Associate Member of the Institution in 1912, and was transferred to the class of Member in 1933. He was a Member of Council from 1942 until 1947.

He was also a Member of the Institution of Mechanical Engineers, and a Fellow of the Royal Institution of Chartered Surveyors.

He leaves a widow, son, and daughter.

CORRIGENDUM

Proceedings, Part I, July 1952

Fig. 6, facing p. 395, *for* "H 10 WP" *read* "H 15 WP"

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